EXHIBIT 7.9

LESSALT WTP PROCESS ALTERNATIVES EVALUATION TM, 2012

LESSALT WTP PROCESS ALTERNATIVES EVALUATION

Hollister Master Plan Implementation Program ***February 13, 2012***

Introduction

The Lessalt Water Treatment Plant (WTP) is currently operating at a maximum hydraulic capacity of around 2 million gallons per day (MGD), but requires plant improvements to meet regulatory requirements and water quality objectives concerning levels of disinfection-by-products (DBPs) and iron (Fe) and manganese (Mn). The draft *Preliminary Design Engineering Report for the Lessalt WTP (DBP Reduction Improvements Project)* (PDR, Kennedy Jenks, June 2011) recommended improvements to meet these water quality objectives and increase the plant hydraulic capacity to 3 MGD. The PDR estimated an overall project cost of approximately $12.5 million to implement these improvements to 3 MGD capacity. However, preliminary financial analyses indicated that a phased approach with lower initial capital costs and operating capacity would be more affordable.

The objective of this Technical Memorandum (TM) is to perform a screening evaluation and re-evaluate lifecycle costs on the current proposed and alternative treatment processes. The evaluation and recommendations are based on an operating capacity of 2 MGD. All evaluated Lessalt WTP alternatives will meet pathogen log-removal requirements and water quality (and regulatory) objectives of reducing Fe, Mn, and DBPs in addition to other basis of design assumptions, as described below.

Background

The Lessalt WTP is supplied by water from the Hollister Conduit, which is a large diameter pipeline that conveys Central Valley Project (CVP) water from San Luis Reservoir (SLR) to San Benito County, where it is used for irrigation and municipal and industrial (M&I) purposes, or stored in the San Justo Reservoir (SJR), a terminal storage reservoir. Figure 1 shows an overview of the source water for the Lessalt WTP.

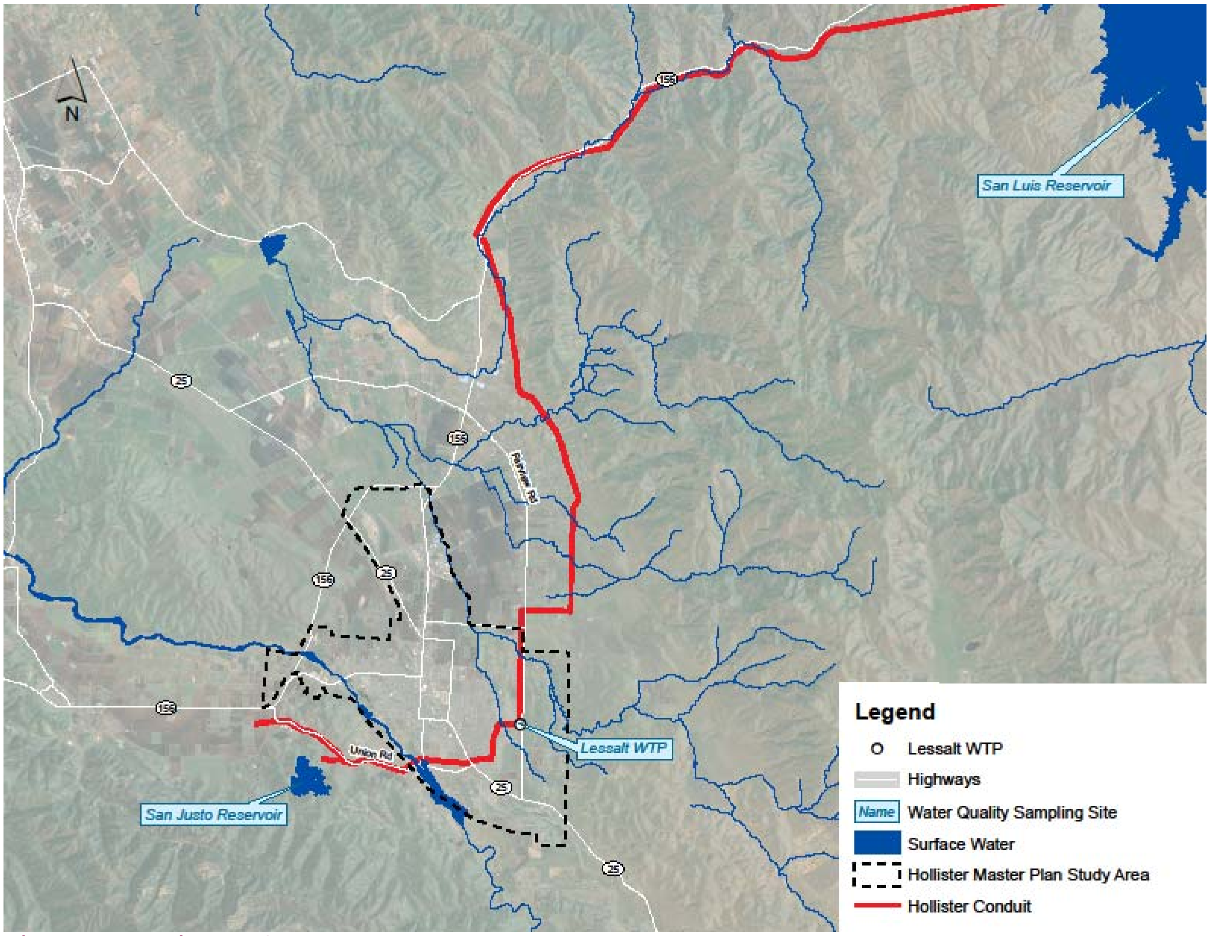


Figure 1. Lessalt WTP Source Water

During most of the year, the primary source of water supply is from the SLR. However, during peak agriculture use periods the Hollister Conduit is back fed from SJR, and thus the source water quality changes. Thermal stratification in SJR during the summer leads to near-anoxic conditions in the lower levels of the reservoir (hypolimnion), where the reservoir outlet to the conduit is located. Anoxic conditions promote the dissolution and release of reduced Fe and Mn from the reservoir sediment, leading to an increase in the SJR source water. Because there is currently no method of Fe and Mn removal at the plant, the Lessalt WTP shuts down when SJR provides the primary source water due to high Fe and Mn concentrations.

The current treatment processes at the Lessalt WTP are microfiltration and free-chlorine disinfection. To allow the WTP to continuously operate using SJR source water, the Kennedy-Jenks PDR proposed improvements for Fe and Mn removal that consist of an oxidant feed system followed by microfiltration membrane separation. In addition, the plant will also be facing challenges to reliably meet the Stage 2 Disinfectants and Disinfection Byproducts (D/DBP) Rule. As stated in the PDR, proposed improvements for reducing DBPs consist of a coagulant feed system followed by microfiltration and nanofiltration membrane separation to reduce total organic carbon (TOC) and subsequently, reactive natural organic matter (NOM), a DBP-precursor.

Water Quality Review

A review of water quality regulations and source water quality was described previously in the *Process Alternatives Screening Evaluation* (August 2010). However, some new information from 2009 and 2010 regarding raw water manganese from the San Justo Reservoir was recently obtained.

San Justo Reservoir Water Quality

Because San Justo Reservoir water originates in San Luis Reservoir, the water quality is presumed to be very similar in most respects. However, water quality data from San Justo Reservoir is limited because, as previously described, the Lessalt WTP is not operated during periods when water in the Hollister Conduit is being back fed from San Justo, due to the presence of Mn which creates challenges.

Although more extensive raw water quality data should be collected from SJR to better characterize the water quality, some data was provided on raw water Mn in 2009 and 2010. In 2009, there was a combined three month period (primarily in the late summer) when the Lessalt WTP was back fed by SJR, where Mn in the raw water ranged from 0.25 to 0.4 mg/L. In 2010, the Lessalt WTP was on SJR for a combined period of about 1.5 months, where Mn levels ranged from 0.10 to 0.18 mg/L.

Water Quality Objectives

The proposed preliminary treated water quality objectives, presented in Table 1, were developed based on known raw water quality data from SLR, maximum contaminant level goals (MCLGs), secondary MCLs, EPA and CDPH standards (i.e., LT2SWTR, LT1SWTR, Stage 2 D/DBP rule, etc.) and use of free-chlorine for disinfection.

High Mn levels in the raw water must be reduced for aesthetic reasons. The proposed goal for treated water manganese concentration is 0.02 mg/L (secondary MCL of 0.05 mg/L).

Raw water TOC levels provide an important role in quantifying the amount of NOM in the water source. TOC in source water comes from NOM and synthetic sources. When the raw water is chlorinated, active chlorine compounds react with NOM to produce chlorinated DBPs. Based on the results of the bench scale jar testing and simulated distribution system (SDS) testing that was completed for the *Preliminary Design Engineering Report of the Lessalt Water Treatment Plant Disinfection Byproduct Reduction Project* (September 2006), disinfection with free chlorine requires that TOC levels be reduced below 1.5 mg/L. The ultimate TOC concentration goal may be adjusted based on a variety of variables such as bromide concentration, distribution system water age, disinfection practices, SDS testing, etc. The treated water bromide concentration is also a factor in DBP formation; however, targeting TOC removal is a more effective solution because it encompasses a broad range of constituents.

Table 1. Proposed Preliminary Treated Water Quality Goals

Parameter	Units	Primary MCL	Secondary MCL	Treated Water Goal
Turbidity	NTU	TT[a]	-	<0.1
Manganese	mg/L	-	0.05	<0.02
Iron	mg/L	-	0.3	<0.1
Disinfection	mg/L-min	-	-	Exceed CT requirement
Distribution System Chlorine Residual (as Cl_2)	mg/L	4.0	-	>1.0
Total Coliform	cfu	5% of samples	-	0
Color	pcu	-	15	<10
Odor	ton	-	3	2.4 (80% of secondary MCL)
TDS	mg/L	-	500	<500
CaCO3 Precipitation Potential	mg/L as CaCO3	-	-	2 to 10
pH	-	-	6.5-8.5	Approx. 8.0
TOC	mg/L	-	-	<1.5 [b]
TTHM at 14 days	mg/L	0.080	-	.060 (80% of req't)
HAA5 at 14 days	mg/L	0.060	-	.045 (80% of req't)

Note: [a] Treatment technique.
[b] TOC concentration goal may be adjusted depending on other variables and constituents.

Process Alternatives

This section discusses the screening evaluation of treatment process alternatives for the Lessalt WTP based on the water quality objectives described in the previous section. The primary objectives for selecting a treatment process are to:

- Produce water at the Lessalt WTP with a capacity of 2 MGD.
- Achieve consistent compliance with WQ objectives.
- Construct and operate economically and feasibly.

The goal of the process alternative screening is to identify a preferred treatment process for the Lessalt WTP.

Primary Treatment Concerns

Although several processes can produce treated water meeting or exceeding regulatory requirements, the parameters most influencing process selection for the Lessalt WTP are DBPs and Fe and Mn. DBPs result from the use of chlorine as the primary and secondary disinfectant at the WTP. Fe and Mn result from the primary use of SJR as the source of raw water during seasonal release events.

There are two common approaches to reduce total trihalomethanes (TTHM) and five haloacetic acids (HAA5) formation, including 1) removal or reduction of the naturally occurring organic precursors that form DBPs when reacted with chlorine, and 2) using an alternative disinfectant which minimizes the formation of DBPs, such as chloramines or chlorine dioxide. Only the first approach was investigated for this evaluation.

There are also three approaches to reducing Fe and Mn, including 1) removal or reduction by physical or chemical treatment at the Lessalt WTP; or 2) reduction by oxygenating or aerating the raw water source off-site; or 3) reduction by operationally adjusting or varying the source water supply. All three approaches were investigated for this memo.

Basis of Design Assumptions

Prior to defining and evaluating process alternatives to meet the primary treatment concerns described above, additional basis of design assumptions were defined and agreed upon. These basis of design criteria were used to narrow the pool of feasible treatment options for evaluation. The agreed upon basis of design criteria include:

- 2 MGD design capacity.
- Meet pathogen log-removal requirements.
- Meet the DBP Rule.
- Meet the secondary MCL for Fe and Mn.
- Continue to filter both SJR and SLR source water.
- Ensure that SJR water can be periodically used as the primary source.
- Ensure the processes are structurally reliable.
- The plant shall operate as a base load water supply for the distribution system.
- Ensure reliable pressure for both supply and demand.
- Allow for future plant expansion / upgrade flexibility.
- Ensure reliable electrical supply.

Major Process Alternatives

From the primary treatment concerns and basis of design assumptions, three major process alternatives were developed. The process technologies which make up these alternatives are briefly described in Table 2.

The three major process alternatives are presented schematically and described briefly in the following subsections. These major process alternatives each include an iron and manganese removal process at the plant. However, low iron and manganese in the source water can be reliably achieved using options described later in the *Source Water Options* section.

Table 2. Treatment Process Descriptions

Treatment Process Technology	Description
Oxidation	Removal of Fe and Mn from the water can be achieved through oxidation with potassium permanganate followed by removal through filtration. Potassium permanganate oxidizes Fe and Mn into their insoluble states. The dose must be great enough to oxidize all of the Mn, but not too great as this will produce a pink color in the water in the distribution system. Use of permanganate is more effective at oxidizing Mn than aeration or chlorination.
Coagulation	A coagulant type and dose is assumed to optimize solids and DBP precursor removal. Enhanced coagulation is the practice of increasing the dose of coagulant beyond the level needed to optimize filtration in order to achieve supplemental removal of DBP precursor materials (organics).
Greensand and Anthracite Roughing Filters	Following oxidation, Fe and Mn are in particulate form and can be removed by downstream filters. Greensand media filters facilitate the adsorption and catalytic oxidation of Mn on greensand media. Anthracite media used in roughing filters removes coagulated solids.
Microfiltration and Nanofiltration Membranes	Membranes serve as a physical barrier against pathogens and some viruses. Membranes separate substances from feed water through sieving actions using various pore sizes and operating pressures. Microfiltration, a low-pressure membrane process, has the largest pore size and can remove *Giardia lamblia* cysts and *Cryptosporidium* oocysts as well as other microorganisms, colloids, and high-molecular weight compounds. Nanofiltration operates at a much higher pressure and smaller pore size and is capable of removing hardness, pathogens, viruses, some total and dissolved organic carbon, and organic color.
GAC Adsorption	Granular activated carbon (GAC) is an adsorption medium that removes elements from a water stream by adsorbing to its porous surface. Depending on design criteria and replacement frequency, GAC can be used for the removal of disinfection byproduct precursors and/or taste and odor compounds.

Alternative 1 – Microfiltration and Nanofiltration

As illustrated in Figure 2, Alternative 1 consists of chemical oxidation and coagulation followed by microfiltration membranes for removal of coagulated solids and oxidized Fe and Mn. Some portion of the membrane filtrate would then enter nanofiltration membranes for additional total and dissolved organic carbon removal. Free chlorine is added prior to the clearwell for distribution system residual. Optional spray aeration into the clearwell could help reduce DBP formation. Both microfiltration and nanofiltration membrane clean-in-place (CIP) chemicals would be neutralized and sent to the sanitary sewer. Although all of the microfiltration membrane backwash would be sent to the sanitary sewer, some portion of the nanofiltration concentrate could be sent back to the Hollister Conduit.

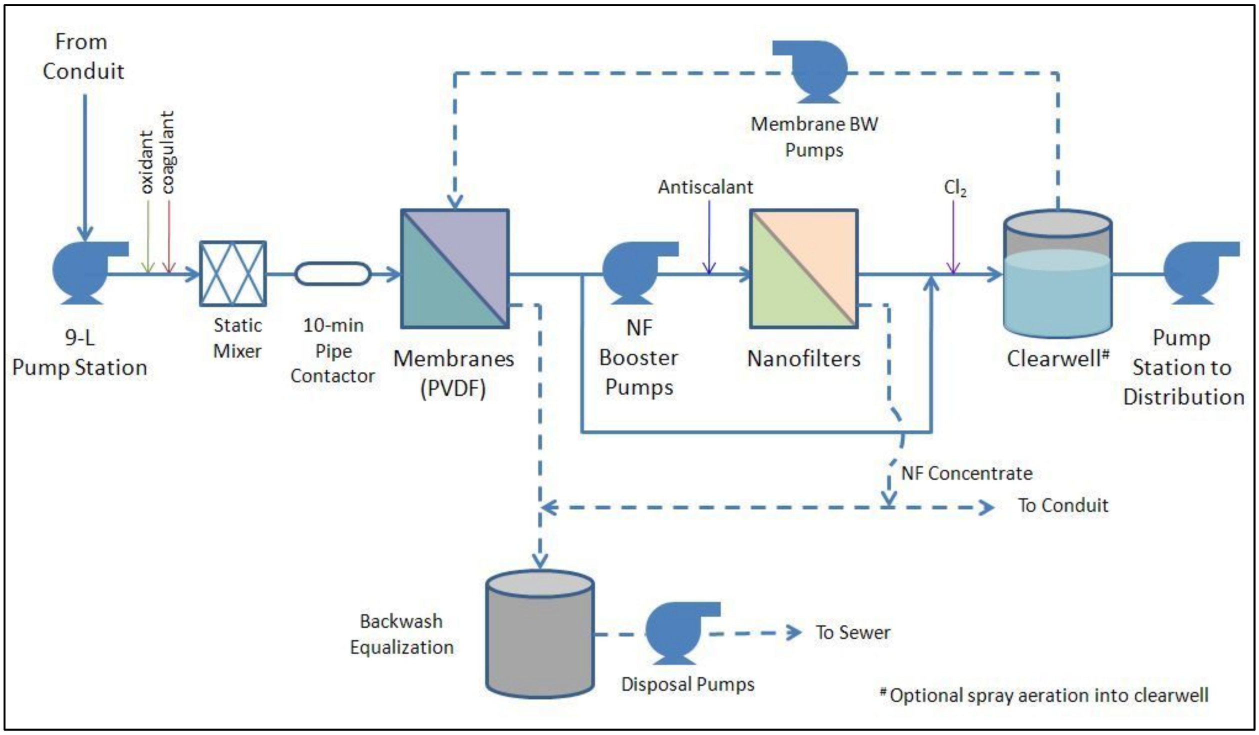


Figure 2. Alternative 1 Process Schematic

Several modifications to the existing plant are necessary for Alternative 1. Oxidation (potassium permanganate), coagulation (polyaluminum chloride or ACH), and anti-scalant chemical systems are required. To provide contact time for oxidant and coagulant, a large diameter (48-inch) pipe contactor would be installed in the yard upstream of the microfiltration membranes. The existing polypropylene (PP) membranes at the Lessalt WTP would require an upgrade to oxidant-resistant polyvinylidene fluoride (PVDF) membranes, entailing modification to the existing skid configuration. For nanofiltration, a new building would be needed to house the nanofiltration membranes, NF booster pumps, and NF cleaning system. The NF building could also contain oxidant and coagulant chemical storage and house the chemical feed systems.

Alternative 2 – Greensand/ Anthracite Filtration, GAC, and Microfiltration

Alternative 2, illustrated in Figure 3, consists of pressurized greensand/ anthracite roughing filters to remove iron and manganese, GAC contactors to remove total and dissolved organic carbon, and the existing microfiltration membranes to remove solids and provide an additional barrier during the demonstration period. After an expected maximum two years of demonstrated removal, regulatory approval of 2-stage filtration (greensand/ anthracite filtration and GAC) would allow the existing microfiltration membranes to be taken offline. Preliminary discussions with the California Department of Public Health suggest that pathogen removal credits and regulatory approval for a Greensand/ Anthracite + GAC treatment train could be obtained by considering the GAC filter media the primary treatment process. Currently, there are several plants in California that have received regulatory approval using GAC filter media, including the Yucaipa Valley Water District.

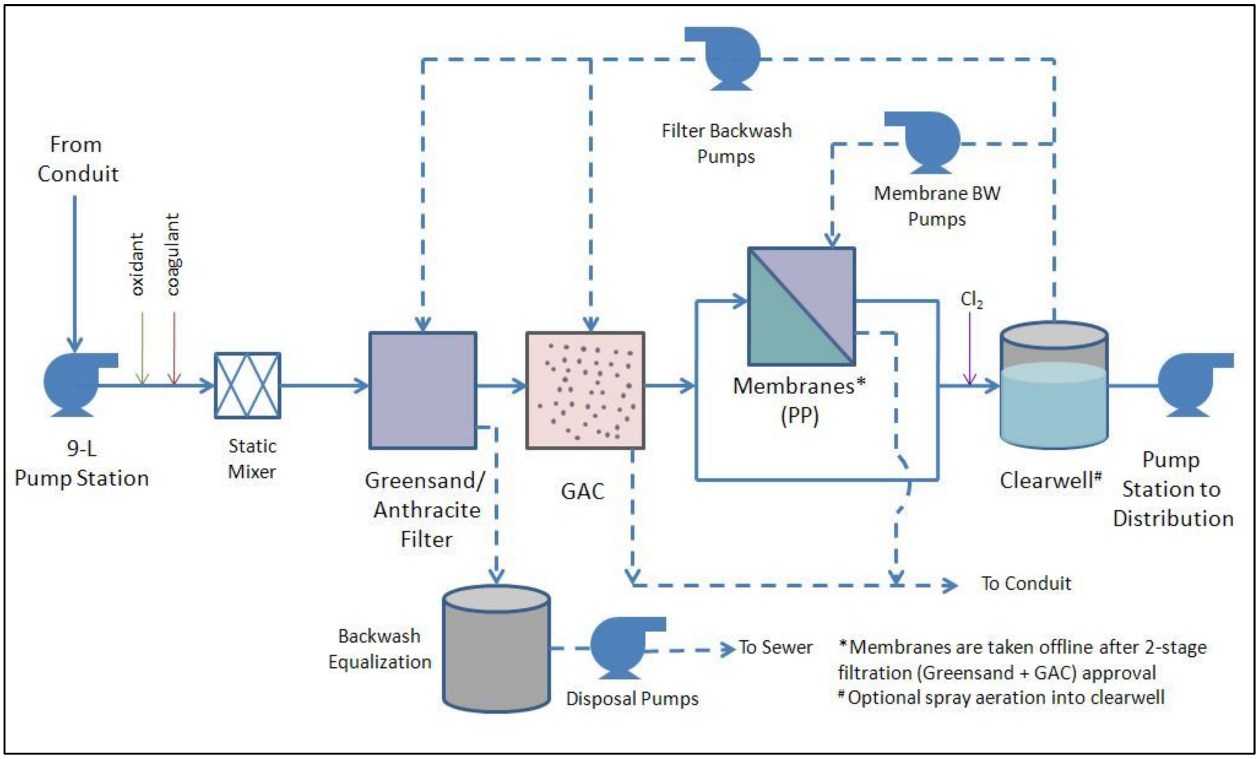


Figure 3. Alternative 2 Process Schematic

Alternative 2 places greensand roughing filters and GAC contactors upstream of the microfiltration membranes. A variation on this alternative, where microfiltration is upstream of Fe and Mn removal is not recommended because of the potential for Fe and Mn fouling on the MF membranes. Free chlorine is added prior to the clearwell for distribution system residual. Optional spray aeration into the clearwell could help reduce DBP formation. Backwash from the greensand/ anthracite filters would need to be sent to the sanitary sewer because oxidant and/or coagulant would be present in the filtrate. Alternative 2 would allow backwash from GAC and membranes to be sent back to the Hollister Conduit. CIP cleaning chemicals for the membranes require neutralization before being sent to the sanitary sewer.

Alternative 2 would also require several modifications to the existing plant. Oxidation (potassium permanganate) and coagulation (polyaluminum chloride or ACH) chemical systems are required and would require a new, dedicated storage and feed area. Large horizontal first-stage pressure filter tanks containing greensand and anthracite media would be installed in an open area outside of the membrane building. Large horizontal second-stage pressure tanks containing GAC media would be installed next to the first-stage roughing filters. No modification of the existing PP microfiltration membranes or skids would be required for this option, as no oxidant would be expected in the microfiltration feed.

Alternative 3 – Microfiltration and GAC

Alternative 3, shown in Figure 4, consists of oxidation and coagulation, microfiltration membranes, GAC contactors, and free chlorine for distribution system residual. Iron and manages would be removed through the combination of chemical oxidation and physical separation of the precipitated solids at the microfiltration membranes. Total and dissolved organic carbon would be removed at the GAC contactors. Free chlorine is added prior to the clearwell for distribution system residual. Again, optional spray aeration into the clearwell could help mitigate DBP levels. Backwash from the membranes would be sent to the sanitary sewer and backwash from the GAC contactors would be sent to the conduit. CIP cleaning chemicals for the membranes would be neutralized and sent to the sewer.

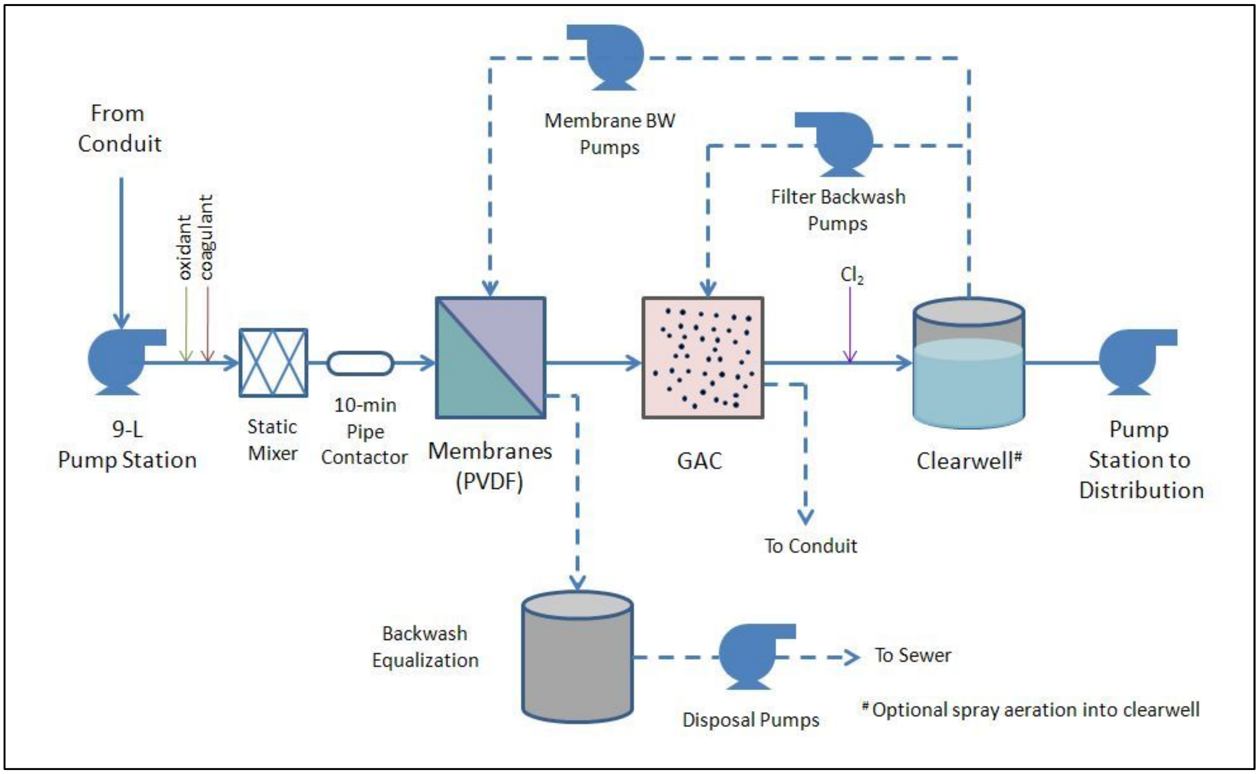


Figure 4. Alternative 3 Process Schematic

Alternative 3 would require several modifications to the existing plant. Oxidation (potassium permanganate) and coagulation (polyaluminum chloride or ACH) chemical systems would require a new, dedicated storage and feed area. To provide contact time for oxidant and/or coagulant, a large diameter (48-inch) pipe contactor would be installed in the yard upstream of the microfiltration membranes. The existing polypropylene (PP) membranes and skids at the Lessalt WTP would require upgrade to oxidant-resistant polyvinylidene fluoride (PVDF) membranes and skids. Large horizontal pressure tanks containing GAC media would be installed downstream of the microfiltration system.

Source Water Options

In addition to the three major process alternatives, there are several options that could reduce or replace the need for Fe and Mn treatment at the Lessalt WTP. The first three options involve treatment or conditioning of the source water from SJR and the fourth option requires operationally controlling the amount of SJR water entering the Lessalt WTP.

There is an ongoing problem with invasive zebra mussels in SJR, potentially threatening the Hollister Conduit. Currently, the SBCWD controls the spread of zebra mussels by maintaining low dissolved oxygen in the system. Therefore, any aeration or oxygenation option in the reservoir or Hollister Conduit could not be implemented until after the invasive species are successfully eradicated. The SBCWD is currently planning a zebra mussel eradication program using potash treatment which may take two or three years to fully implement. If the eradication program is successful, the oxygenation options below would be a feasible alternative to chemical and physical treatment at the plant.

The four source water options are illustrated in Figure 5 and described below.

Option 1: Reservoir Mixing

Floating aerators or mixers (i.e., as manufactured by Wears) can be installed at the reservoir surface near the Hollister Conduit outlet to mix the hypolimnion during periods of thermal stratification and seasonal Fe and Mn release. The mixing will maintain high dissolved oxygen levels near the reservoir bottom and reduce the dissolution of Fe and Mn from sediment. The mixers would reduce anoxic conditions in the reservoir and help control taste and odor. These mixers could also be used to target and disseminate chemical to eradicate zebra mussels; however, the reliability and effectiveness of this strategy is subject to many unknown variables and is highly risky.

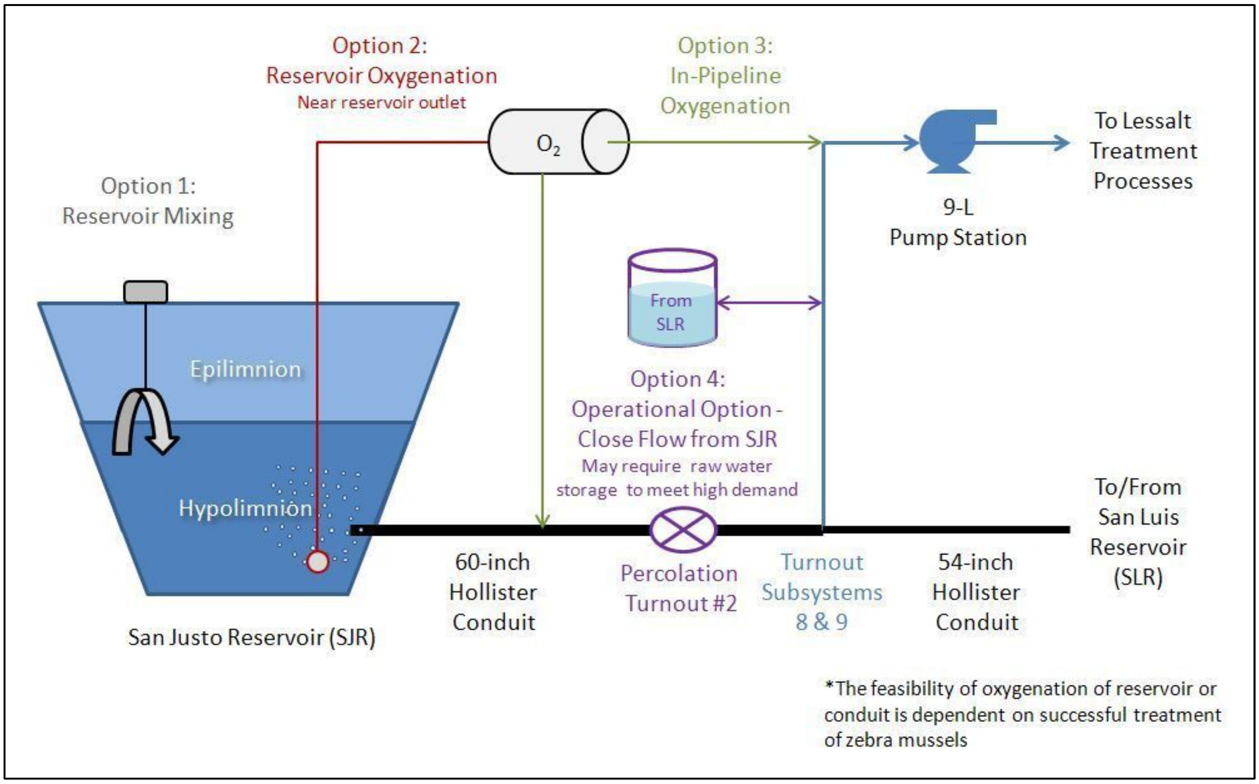


Figure 5. Source Water Options for Fe and Mn

Option 2: Reservoir Oxygenation

Reservoir oxygenation can be accomplished by installing oxygenation tubes that sit near the reservoir bottom (i.e. as designed and manufactured by Mobley Engineering). By targeting oxygenation of the sediments near the reservoir outlet, the amount of Fe and Mn entering the Hollister Conduit could be reduced. These tubes are mobile and could be used to disseminate chemicals to eradicate zebra mussels, although the reliability of using this method for mussel control is subject to several unknown variables.

Option 3: In-Pipeline Oxygenation

Oxygenation could also be applied to the raw water in the pipeline, either in the large diameter Conduit or in the raw water influent line to the plant. Pipeline oxygenation or aeration would require Fe and Mn removal at the plant if the oxidized Fe and Mn is not removed off-site.

Option 4: Operational Option

The operational option consists of installation of a large diameter valve on the 60-inch Hollister Conduit at Percolation Turnout #2 (PT2). During periods of high iron and manganese from

SJR, the valve at the PT2 would close. After the valve is closed, all demands between the Pacheco bifurcation structure and PT2 (and including the Lessalt WTP) would be supplied by water from SLR, and all demands between PT2 and San Juan sublateral would be supplied by SJR.

Screening of Source Water Options

Costs of the four source water options were investigated (refer to Appendix C) but the options were all ultimately rendered infeasible and rejected for various reasons.

Although the costs associated with Options 1 and 2 are comparable to the costs of treating Fe/ Mn at the WTP, the presence of zebra mussels complicates implementation of this strategy.

Option 3 would require pipeline oxygenation on the raw water line at the plant site, to prevent the migration and establishment of mussels in the Conduit. In-pipeline oxygenation at the WTP would still necessitate infrastructure for Fe/ Mn removal, and the treatment costs associated with this option are comparable to treatment costs associated with using chemical oxidant (i.e., permanganate). Chemical oxidation/ treatment at the plant is recommended over Option 3 because oxygenation/ aeration of Mn requires long (greater than 1 hour) detention times to be effective.

Option 4 requires operationally managing the source water entering the Lessalt WTP. When Fe/ Mn concentrations are high, a valve on the Hollister Conduit near PT#2 would be closed to restrict flow from SJR, allowing only water from SLR to feed the plant. Historically, SJR is the primary source of water for the Lessalt WTP for a period of up to 3 months. This period often overlaps with the high demand late summer/ early fall months. Preliminary modeling results suggest that closing the valve at PT#2 for 3 months during high demand summer periods would prevent demands to be met at the turnout locations along the Hollister Conduit at the pressures required.

Screening Criteria

Each of the three major process alternatives were evaluated using the cost and non-cost screening criteria described below. Alternatives were ranked according to how well they meet each criterion. For the non-cost criteria, the “High” criteria ranking is most favorable and the “Low” criteria ranking is least favorable. Criteria 1 through 6 are listed in approximate decreasing order of relative importance, based on the priorities of the MOU Parties. The weighting factor used in the scoring evaluation is also listed for each criterion.

Criterion 1: Project Capital Costs – 25% Weight Factor

This criterion includes the opinion of capital cost for the process alternatives and is weighted 25% in the evaluation. The capital costs for the alternatives include an opinion of probable construction cost for the treatment processes and related support infrastructure for the process alternative.

Criterion 2: Project Present Worth Costs – 25% Weight Factor

This criterion includes the opinion of present worth life-cycle costs for the process alternatives and is weighted 25% in the evaluation. The present worth cost was developed based on a present worth life-cycle cost analysis of the capital cost, annual O&M cost, and major MF membrane replacement cost, as described below.

- **Capital Costs** – The capital costs for the alternatives include an opinion of probable construction cost for the treatment processes and related support infrastructure for the process alternative, the same as described in Criterion 1.
- **O&M Costs** – The O&M costs include energy, chemicals, residuals handling and disposal, membrane, resin or filter media replacement, maintenance materials, and labor. An incremental raw water cost based on the annual cost associated with purchasing raw water above 2 MGD production capacity is also included.
- **Major MF Membrane O&M Costs** – The major MF membrane O&M cost is the cost of replacing the existing MF membrane system components (i.e., valves, electronics, etc.) at Year 10, when the existing MF systems will near the end of an assumed 20-year lifespan.

A 20 year life cycle and 3 percent discount rate are assumed for calculating present worth life-cycle costs. Detailed cost sheets are included in Appendix A.

Criterion 3: Additional Peak Production Capacity – 20% Weight Factor

This performance criterion considers the flexibility of upgrading the proposed treatment alternatives to increase production capacity up to 2.5 MGD for extended periods (e.g., up to a month in duration). Additional water production is desired during high demand periods, seasonal peaking, or when the West Hills WTP is offline. This criterion is weighted 20% in the evaluation. The processes are ranked according to the following performance criteria.

- **High –** Increasing plant capacity should not require modification to the proposed treatment systems, and operations and maintenance will remain the same or slightly increase.
- **Medium –** Increasing plant capacity requires some modification to the proposed treatment systems, and operations and maintenance will slightly increase.
- **Low –** Increasing plant capacity requires major modifications to the existing systems, and operations and maintenance will increase.

Criterion 4: Future Expansion Flexibility - 10% Weight Factor

This performance criterion considers the flexibility of expanding the proposed treatment plant in the future. This criterion includes the cost differential to expand the plant to 3 MGD, the ability to fit future processes and infrastructure on the existing site, and the flexibility to tie in future processes and infrastructure to the proposed alternatives.

- **High –** Future expansion requires minimal modification and costs to the proposed treatment systems, minimal addition or modification to the existing site, and operations and maintenance will remain the same or slightly increase.
- **Medium** – Future expansion requires moderate modification and costs to the proposed treatment systems, some modification or addition to the existing site plan, and operations and maintenance will moderately increase.
- **Low –** Future expansion requires major modifications and costs to the existing treatment systems, major modifications to the existing site or additional land acquisition, and operations and maintenance will increase substantially.

Future expansion flexibility is somewhat contingent on the outcome of a lot line adjustment. The development of Santana Ranch to the north of the site will require 0.5 acres to accommodate the Ranch's access road. This will reduce the Lessalt WTP site from approximately 1.5 acres to 1 acre and shrink the available space for new process construction. Variances from San Benito County setback requirements may be required in order to develop the Lessalt WTP and fit the suggested process alternatives at the site.

Criterion 5: Operational Complexity – 10% Weight Factor

This performance criterion considers the operational complexity of the proposed treatment processes. Operational complexity includes:

- Number of different processes and equipment,
- Number of different chemicals required, chemical hazards, and difficulty in the handling of the chemicals,
- Amount of liquid and solid residuals and associated issues regarding the capture, handling, treatment and disposal of the residuals,
- Level of automation, and
- Ease of operation of the proposed facilities.

The processes are ranked according to the following performance criteria.

- **High** – The processes have a relatively low degree of operational complexity compared to other alternatives.
- **Medium** – The processes have moderate degree of operational complexity compared to other alternatives.
- **Low** – The processes have a higher degree of complexity compared to other alternatives.

Criterion 6: Site Compatibility and Environmental Factors – 10% Weight Factor

This performance criterion considers the environmental factors related to the proposed treatment alternatives. Environmental factors include:

- Electrical energy,
- Chemical delivery and solids disposal truck traffic,
- Aesthetics of the site,
- Compatibility with hydraulics and space at the existing site,
- Aesthetics of new construction at the site.

The processes are ranked according to the following performance criteria.

- **High** – The processes will have lower energy, chemical, and environmental impact and require minor additional space and configuration changes to the Lessalt WTP compared to other alternatives.
- **Medium** – The processes will have moderate energy, chemical, and environmental impact and require some additional space and moderate configuration changes to the Lessalt WTP compared to other alternatives.
- **Low** – The processes will have greater energy, chemical, and environmental impacts and require significant configuration changes to the Lessalt WTP compared to other alternatives.

Process Alternative Screening

Table 3 presents a summary of the process alternatives screening evaluation. As shown in Table 3, and as further described in the subsections below, the alternatives were ranked for how they meet each criterion. Alternatives were ranked using the High/Medium/Low (H/M/L) scale described in the previous section. To distinguish the differences between two or more alternatives with the same ranking, +/-'s were used. The H/M/L ranking was then converted to a numerical ranking as follows:

- H = 5
- M+ = 4
- M = 3
- M- = 2
- L = 1

The numerical rankings for Total Capital Costs and Total Present Worth Costs in Table 3 were assigned by assuming a high (5) score for the lowest cost option, and prorating the costs of the higher cost alternatives.

The screening criteria were weighted to reflect their relative importance to one another; the weighting factors are also shown in Table 3. The evaluation results were multiplied by the weighting factor and normalized to 100.

Table 3. Lessalt WTP Process Alternatives Screening Evaluation

	Economic Criteria			Non-Economic Criteria						
	1	2		3	4	5	6			
Alternative[a]	Total Capital Costs[b]	Total Present Worth Costs[b]	Subtotal	Additional Peak Production Capacity	Future Expansion Flexibility	Operational Complexity & Waste Disposal	Site Compatibility & Environmental Factors	Subtotal	Total	Rank
Evaluation Results										
1 – Microfiltration/ Nanofiltration[c]	-	-	-	M-	M-	M	M-	-	-	-
2 – Greensand Filtration/ GAC/ (Microfiltration)[d]	-	-	-	H	M+	H	M+	-	-	-
2+MF – Greensand Filtration/ GAC/ Microfiltration[e]	-	-	-	M	M-	M+	M	-	-	-
3 – Microfiltration/ GAC[c]	-	-	-	M-	M	M	M+	-	-	-
Evaluation Results										
1 – Microfiltration/ Nanofiltration[c]	4.6	4.9	9.5	2	2	3	2	9	18.5	4
2 – Greensand Filtration/ GAC/ (Microfiltration)[d]	4.9	5.0	9.9	5	4	5	4	18	27.9	1
2+MF – Greensand Filtration/ GAC/ Microfiltration[e]	4.8	4.9	9.7	3	2	4	3	12	21.7	3
3 – Microfiltration/ GAC[c]	5.0	4.9	9.9	2	3	3	4	12	21.9	2
Weighting Factor	25%	25%	50%	20%	10%	10%	10%	50%	100%	
Weighted Evaluation Results[F]										
1 – Microfiltration/ Nanofiltration[c]	23.0	24.5	47.5	8.0	4.0	6.0	4.0	22.0	69.5	4
2 – Greensand Filtration/ GAC/ (Microfiltration)[d]	24.5	25.0	49.5	20.0	8.0	10.0	8.0	46.0	95.5	1
2+MF – Greensand Filtration/ GAC/ Microfiltration[e]	24.0	24.5	48.5	12.0	4.0	8.0	6.0	30.0	78.5	2
3 – Microfiltration/ GAC[c]	25.0	24.5	49.5	8.0	6.0	6.0	8.0	28.0	77.5	3

Notes:

a. Alternatives were initially ranked using a High/Medium/Low (H/M/L) scale. To distinguish the differences between two or more alternatives with the same ranking, +/-'s were used. The H/M/L ranking was converted to a numerical ranking. The numerical ranking for Project Costs was assigned based on prorated present worth costs. The remaining numerical ranking for the remaining criteria was assigned as follows: H = 5, M+ = 4, M = 3, M- = 2, L = 1.
b. Based on capital and present worth cost of 2 MGD capacity. The lowest cost alternative was assigned a score of 5, and other alternatives were assigned a prorated score.
c. Alternatives 1 and 3 require microfiltration membrane upgrade from PP to oxidant-resistant PVDF.
d. Following regulatory approval of 2-stage filtration system, the microfiltration membranes would be decommissioned.
e. Sensitivity analysis to evaluate the relative ranking Alternative 2 if the the MF membranes remain in service.
f. Evaluation results were multiplied by the weighting factor then normalized to 100.

Project Costs

Capital, operation and maintenance, and present value costs were developed to compare the alternatives as part of the screening evaluation. The costs are presented in Table 4 and are based on a discount rate of 3 percent for a period of 20 years. Costs are based on 2012 dollars.

Table 4. Lessalt WTP Process Alternatives Cost Comparison

Process Alternatives	WTP at 2 MGD Capacity			
	Capital Cost[a] ($M)	Annual O&M Cost ($M)	Major Replacement Cost ($M)	Present Value Cost ($M)
1 – Microfiltration/ Nanofiltration	$7.03	$1.05	$0.58	$23.22
2 – Greensand Filtration/ GAC/ (Microfiltration)[b]	$5.44	Year 1-2: $1.14 Year 3-20: $1.03	N/A	$21.77
2+MF – Greensand Filtration/ GAC/ Microfiltration[c]	$5.95	$1.14	$0.60	$23.51
3 – Microfiltration/ GAC	$5.08	$1.26	$0.58	$24.34

Notes:

a. Capital costs include 25% contingency (5% more than previous estimates) due to the uncertainty assocated with the lot line adjustment and potential need for retaining walls to maximize the usable space on the existing site.
b. Following regulatory approval of the 2-stage filtration system, the microfiltration membranes would be decommissioned.
c. Microfiltration membranes remain in service.

Considering that regulatory approval is required before the microfiltration membranes can be removed from the Alternative 2 process, a sensitivity analysis was performed to evaluate the costs two ways: 1) assuming that MF membranes are decommissioned after a 2-year demonstration period (“Alternative 2”) as planned, and 2) assuming that the MF membranes remain in service indefinitely (“Alternative 2+MF”). As shown in Table 4, the capital cost of Alternative 2+MF is higher than Alternative 2 because Alternative 2+MF includes the cost of replacing all the existing PP modules (currently near the end of their useful life) with new PP modules. The cost would be unnecessary for Alternative 2 since the MF membranes would be decommissioned after only two years. Similarly, the annual O&M cost for Alternative 2+MF is slightly higher due to the additional costs associated with operating the MF membranes.

A major replacement cost was added to Alternatives 1, 2+MF, and 3 to address rehabilitation and replacement required for the existing MF system in the tenth year of operation. The existing MF system has an expected 20-year lifecycle, and the skids have already been in operation for almost 10 years. Although the MF skids would be modified and updated to PVDF (for Alternative 1 and 3) or PP (for Alternative 2+MF) modules, other system appurtenances such as valving, electronics, etc. would not be updated. These items would near the end of their useful life after another 10 years of operation. The 10-year replacement cost captures the major renewal needs and were assumed to be approximately 50% of the cost of a new MF skid, and differs slightly for PP and PVDF skids. This cost was not applied to Alternative 2 because MF membranes would be decommissioned before major maintenance on the MF system is required.

As shown in Table 4, and as reflected in the rankings presented in Table 3, Alternative 3 has the lowest capital cost at $5.08 million, followed by Alternative 2 at $5.44 million. Alternative 2 has the lowest total present value cost at $21.8 million, followed by Alternative 1 at $23.2 million. Alternative 2+MF is in the middle with a $5.95 million capital cost and $23.5 million present value cost.

Additional Peak Production Capacity

Alternative 1 (MF and NF membranes) requires some process and system modification/ addition to meet capacity beyond the design 2 MGD. Membrane systems are designed for a specific flux rate, and increasing flux though the membranes increases membrane fouling, backwashing, and chemical cleaning requirements. Operating at higher than design flux rates may also cause mechanical damage to the membrane. (Note that higher flux rates may be possible during the summer months due to higher permeability of the membranes). Skid modification and replacement of PP membranes with PVDF membranes would reduce MF production capacity. Although the standby MF skid could be used to treat additional capacity, the frequent backwashing (on the order of every half hour) and scheduled cleaning requirements (weekly and monthly) restrict their overall temporal availability if sustained production is desired. The standby NF membrane unit does have a less frequent cleaning regimen (monthly) and could provide additional capacity for longer periods than the MF system, although operationally it would still be advantageous to maintain a standby unit. To meet additional peak production capacity, an additional (new) MF membrane skid, an additional NF membrane skid, and an increase in the pumping, piping, appurtenance, and electronics requirements would be necessary. This Alternative was ranked 2.

Alternative 2 (greensand/ anthracite filtration and GAC) is the most flexible alternative to increase additional treatment capacity. The greensand/ anthracite filter loading rate would increase and the empty bed contact time would decrease to allow for additional treatment capacity without greatly affecting water quality objectives. In addition, less frequent backwash requirements for the greensand/ anthracite filter (compared to the membrane alternatives) may allow the standby filter unit to be used during peak production periods. Loading on the GAC filters would increase with increased capacity, but would affect only the frequency of media change-out and neither require installation of additional hydraulic capacity onto nor reduce the amount of TOC removal. The existing MF membranes have a 2.02 MGD production capacity with the two duty units and would require using the third standby unit to produce up to 2.5 MGD. However, the MF system functions more as a final barrier in the treatment train, and would be decommissioned after a demonstration period for Alternative 2 (and left in place for Alternative 2+MF). No major changes or additions to the base 2 MGD system are required as long as the plant piping and the treated water tank (for adequate disinfection contact time) is initially designed for a slightly higher treatment capacity. Alternative 2 was ranked 5, and Alternative 2+MF was ranked 3.

Alternative 3 (MF membranes and GAC) is moderately flexible, and would likely require installation of a new MF membrane skid and associated appurtenances and electronic programming to meet additional MF filtration capacity (similar to Alternative 1). Like Alternative 2, the GAC system could easily accommodate additional flow and the only major effect would be an increase in the frequency of media replacement. Alternative 3 was ranked 3.

Future Expansion Flexibility

Alternative 1 has the highest cost associated with expansion of facilities to 3 MGD. To meet 3 MGD production, another MF skid could be accommodated in the current process building. However, another NF skid and associated appurtenances could not fit in the proposed NF building and construction of a second NF building would be necessary. Because of the high relative cost of expansion and possibility for additional land acquisition and/or permitting requirements for new NF building construction, Alternative 1 was ranked 2.

Alternative 2 has the lowest cost associated with expansion of facilities to 3 MGD. To meet 3 MGD production, the addition of another greensand/ anthracite filtration unit and another GAC unit would be required. These horizontal tanks could probably fit on existing site adjacent to the proposed tanks, although the space available for expansion would be very limited. Alternative 2 was ranked 4.

Alternative 2+MF, where the MF membranes remain indefinitely, would require an additional PP MF skid to be added in addition to the greensand/ anthracite and GAC filter units to meet 3 MGD capacity. Because of the high additional cost of adding a fourth MF unit to meet production requirements, Alternative 2+MF was ranked 2.

Alternative 3 would require a new membrane skid and additional GAC unit with expansion of facilities to 3 MGD. The cost of expansion is slightly less expensive than Alternative 1, but much more expensive than Alternative 2. However, Alternative 3 requires the least additional land development and has the most room for expansion to 3 MGD and beyond. Alternative 3 was ranked 3.

For all alternatives, future expansion would be easier if additional space was available (e.g., through a lot line adjustment to the south).

Operational Complexity

All alternatives that include membrane filtration are highly automated, but are also more operationally complex than their non-membrane counterparts. To prevent membrane fouling, microfiltration requires greater backwash and chemical cleaning requirements than greensand/ anthracite filtration. Membrane filtration alternatives are generally ranked lower than pressurized greensand/ anthracite filtration alternatives in terms of operational ease.

Both NF and GAC processes add some system complexity. The NF system is highly automated but does require the addition of anti-scalant. The maintenance efforts associated with replacement of spent GAC add to the relative operational complexity, although the GAC supplier typically removes and installs the media. Both NF and GAC processes are ranked similarly in operational complexity.

Alternative 1 (the NF and MF processes) requires storage and handling of 6 treatment process chemical systems (oxidant, coagulant, acid, caustic, anti-scalant, and chlorine). This alternative produces the greatest amount of liquid waste disposal. Based on the high chemical requirements, high liquid waste disposal requirements, and the relatively high complexity of these two membrane processes, Alternative 1 was ranked 4.

Alternative 2 (the greensand/anthracite and GAC processes) requires 4 treatment process chemical systems (oxidant, coagulant, GAC, and chlorine) following the decommissioning of

MF membranes. This alternative produces the lowest amount of liquid waste disposal, but will require GAC media change-out disposal few times a year. Based on the chemical requirements, low liquid disposal requirements, and the relatively low complexity of these two filtration processes (compared with membrane systems), Alternative 2 was ranked 5.

Alternative 2+MF, where the MF membranes remain, requires 6 treatment process chemical systems (oxidant, coagulant, GAC, acid, caustic, and chlorine). This alternative has a similar low rate of waste disposal as Alternative 2, since the MF backwash is sent back to the Conduit. Alternative 2+MF is ranked 4, due to the higher chemical requirements and higher system complexity as compared to Alternative 2 without MF.

Alternative 3 (the MF and GAC processes) requires storage and handling of 6 treatment process chemical systems (oxidant, coagulant, acid, caustic, GAC, and chlorine). This alternative is associated with large volumes of liquid waste disposal, second only to Alternative 1. Based on the high chemical requirements, high rates of both liquid and solid disposal, and the moderate complexity of these two processes to others, Alternative 2 was ranked 3.

Site Compatibility and Environmental Factors

Alternatives with membrane filtration consume more energy than their counterparts with pressurized gravity filtration. Therefore, energy-intensive NF and MF processes were generally ranked lower than the alternatives.

Draft proposed site plans (Appendix B) show that all three alternatives can currently fit within existing property lines and will not require additional land acquisition or permitting requirements. As discussed previously, the land to the north of the Lessalt WTP is scheduled to become an access road for a new development, and about 0.5 acres will be lost at the WTP site. Although the Alternatives all fit on the site, a variance from road setback requirements may be required depending on the desired location of process units. From a process configuration, site development, and aesthetics perspective, the three alternatives also differ as described below.

Alternative 1 combines energy-intensive MF and NF membrane feed and maintenance. This alternative also requires large land use and development requirements for new construction of oxidation/ coagulation contactor, NF process building, treated water tank, and distribution system pump stations. However, all Alternative 1 processes can fit on the existing site. The overall system compatibility of proposed processes with the existing system is moderate, and will involve some MF membrane and piping modifications and the installation of intermediary process pumps to provide adequate feed pressure for the new NF membranes. The chemical delivery schedule is expected to be highest for this alternative, corresponding to the highest expected truck delivery traffic. Alternative 1 was ranked 3.

Alternative 2 combines the least energy intensive greensand and GAC processes (after MF filters are decommissioned). Infrastructure in the form of large outdoor horizontal tanks for the greensand filtration and GAC processes can be constructed at the existing site, which may affect overall plant aesthetics. All proposed processes and improvements can fit on the existing site but the alternative requires an increase in site development and land usage (similar to Alternative 1) for new construction of the greensand/ anthracite roughing filters, GAC filters, small chemical storage area, treated water tank, spent washwater tank, and backwash and distribution system pump stations. The overall proposed system compatibility with existing processes is moderate and will require new piping between greensand, GAC, and MF processes. Chemical usage is the lowest of the options, although GAC media will require replacement 3-4 times per year. Alternative 2 was ranked 4.

Alternative 2+MF uses a moderate amount of electrical energy relative to other processes, slightly higher than Alternative 3, and a high amount of expected traffic for chemicals (and GAC) delivery. This alternative can fit on the existing site and has the same footprint and requirements as Alternative 2, but uses the MF building and existing MF membranes rather than eventually decommissioning them. Aesthetically, Alternative 2+MF is the same as Alternative 2. The compatibility with existing processes is also similar to Alternative 2. Alternative 2+MF was ranked 3, slightly lower than Alternative 2 because of environmental (energy and traffic) factors.

Alternative 3 uses a moderate amount of electrical energy relative to other processes. This alternative also requires a moderate amount of chemicals and GAC media replacement 3-4 times per year. Alternative 3 processes can fit on the existing site and require the smallest amount of land development for the construction of an oxidation/ coagulation contactor, small chemical storage area, spent washwater tank, treated water tank, and backwash and distribution system pump station. The overall proposed system compatibility with existing processes is moderate compared to the other processes and will require MF membrane modification and new installation of process piping between the MF and GAC units. Plant aesthetics may be affected by outdoor construction of large outdoor GAC, backwash, and treated water tanks. Alternative 3 was also ranked 4.

Screening Evaluation Conclusions

Because 20-year present worth costs are similar for all three options, other non-cost criteria were critical to separate and recommend the best alternative.

As summarized in Table 3, Alternative 2 (greensand/ anthracite filtration with GAC) was the highest ranked alternative based on cost and non-cost factors, both before and after the weighting factors are applied. If the membranes were to remain in use at the site, Alternative 2+MF would also rank higher than Alternatives 1 and 3.

Alternative 3 (MF membranes with GAC) was ranked behind the greensand/ anthracite filtration with GAC alternatives and has the lowest capital cost but highest overall present worth cost because of high O&M costs.

Alternative 1 (MF membranes with NF membranes) had the highest capital cost, moderate overall operating costs, and was ranked last overall for total present worth costs. The low score for Alternative 1 can be attributed to the low ranking the alternative received for the non-economic criteria, including additional peak production capacity, future expansion flexibility, and site compatibility and environmental factors.

Recommendation

The screening evaluation concluded that Alternative 2, which includes the greensand and GAC treatment processes, best meets the economic and non-economic criteria as defined and weighted in this memo. If the MF membranes remain in service as a final polishing step or pathogen barrier, Alternative 2+MF still ranks higher than Alternative 1 or 3, although the present value cost for the alternative is slightly higher than Alternative 1. Alternative 2 is therefore recommended for further evaluation.

APPENDIX A

Capital and O&M Costs

Capital Costs: Alternative 1, Microfiltration + Nanofiltration

LESSALT WTP IMPROVEMENTS - DBP SOLUTION - for 2 MGD					
Cost Components	**TOTAL**	**Hydraulic Capacity, TW Tank, & Pump Stations**	**Nanofiltration**	**Membrane Replacement**	**Coagulation**
Sitework and Yard Piping	$ 230,000	$ 93,000	$ 93,000	$ 10,000	$ 34,000
Oxidation/ Coagulation Contactor	$ 114,000				$ 114,000
MF System Upgrades / Mn & Fe Reduction System	$ 445,000			$ 445,000	
DBP Reduction System	$ 2,066,000	$ 50,000	$ 2,016,000		
TW Tank and Fairview PS	$ 152,000	$ 152,000			
Return Water System	$ 125,000		$ 125,000		
Chemical Systems	$ 137,000	$ 11,000	$ 33,000	$ 33,000	$ 60,000
Electrical and Instrumentation (17%)	$ 555,000	$ 52,000	$ 385,000	$ 83,000	$ 35,000
Subtotal	$ 3,824,000	$ 358,000	$ 2,652,000	$ 571,000	$ 243,000
Division 1 Costs (@ 8%)	$ 306,000	$ 29,000	$ 212,000	$ 46,000	$ 19,000
Taxes - Material Costs (@ 9.25%)	$ 307,000	$ 29,000	$ 213,000	$ 46,000	$ 19,000
Contractor OH&P (@ 10%)	$ 382,000	$ 36,000	$ 265,000	$ 57,000	$ 24,000
Estimate Contingency (@ 25%)	$ 957,000	$ 90,000	$ 663,000	$ 143,000	$ 61,000
Total Lessalt Bid Estimate	$ 5,776,000	$ 542,000	$ 4,005,000	$ 863,000	$ 366,000
Ridgemark PS Bid Estimate	$ 210,000	$ 210,000			
Design (@ 8%)	$ 461,000	$ 43,000	$ 320,000	$ 69,000	$ 29,000
Construction Mgmt (@ 10%)	$ 578,000	$ 54,000	$ 401,000	$ 86,000	$ 37,000
Subtotal Lessalt Project Estimate	$ 7,025,000	$ 849,000	$ 4,726,000	$ 1,018,000	$ 432,000
Property Cost @ $100K per acre	$ -				
Total Lessalt Project Estimate	**$ 7,025,000**	**$ 849,000**	**$ 4,726,000**	**$ 1,018,000**	**$ 432,000**
Percent		**12%**	**67%**	**14%**	**6%**

Capital Costs: Alternative 2, Greensand/ Anthracite + GAC + (Microfiltration)

LESSALT WTP IMPROVEMENTS - DBP SOLUTION - for 2 MGD						
Cost Components	TOTAL	Hydraulic Capacity, TW Tank, & Pump Stations	GAC System	Greensand Filters (Mn/Fe Removal)	Coagulation	Chemical Storage Building
Sitework and Yard Piping	$ 224,000	$ 93,000	$ 49,000	$ 48,000	$ 34,000	
Oxidation/ Coagulation Contactor	$ -					
Roughing Filter/ Mn & Fe Reduction System	$ 486,000			$ 486,000		
DBP Reduction System	$ 1,152,000	$ 50,000	$ 1,102,000			
TW Tank and Fairview PS	$ 191,000	$ 191,000				
Return Water System	$ 185,000			$ 185,000		
Chemical Systems	$ 186,000	$ 11,000		$ 33,000	$ 60,000	$ 82,000
Electrical and Instrumentation (17%)	$ 413,000	$ 59,000	$ 196,000	$ 128,000	$ 16,000	$ 14,000
Subtotal	$ 2,837,000	$ 404,000	$ 1,347,000	$ 880,000	$ 110,000	$ 96,000
Division 1 Costs (@ 8%)	$ 227,000	$ 32,000	$ 108,000	$ 70,000	$ 9,000	$ 8,000
Taxes - Material Costs (@ 9.25%)	$ 371,000	$ 53,000	$ 176,000	$ 115,000	$ 14,000	$ 13,000
Contractor OH&P (@ 10%)	$ 284,000	$ 40,000	$ 135,000	$ 88,000	$ 11,000	$ 10,000
Estimate Contingency (@ 25%)	$ 710,000	$ 101,000	$ 337,000	$ 220,000	$ 28,000	$ 24,000
Total Lessalt Bid Estimate	$ 4,429,000	$ 630,000	$ 2,103,000	$ 1,373,000	$ 172,000	$ 151,000
Ridgemark PS Bid Estimate	$ 210,000	$ 210,000				
Design (@ 8%)	$ 354,000	$ 50,000	$ 168,000	$ 110,000	$ 14,000	$ 12,000
Construction Mgmt (@ 10%)	$ 442,000	$ 63,000	$ 210,000	$ 137,000	$ 17,000	$ 15,000
Subtotal Lessalt Project Estimate	$ 5,435,000	$ 953,000	$ 2,481,000	$ 1,620,000	$ 203,000	$ 178,000
Property Cost @ $100K per acre	$ -					$ -
Total Lessalt Project Estimate	**$ 5,435,000**	**$ 953,000**	**$ 2,481,000**	**$ 1,620,000**	**$ 203,000**	**$ 178,000**

Percent		**21%**	**55%**	**36%**	**5%**	**4%**

Capital Costs: Alternative 2+MF, Greensand/ Anthracite + GAC + Microfiltration

LESSALT WTP IMPROVEMENTS - DBP SOLUTION - for 2 MGD							
Cost Components	**TOTAL**	**Hydraulic Capacity, TW Tank, & Pump Stations**	**GAC System**	**Greensand Filters (Mn/Fe Removal)**	**Membrane Replacement**	**Coagulation**	**Chemical Storage Building**
Sitework and Yard Piping	$ 224,000	$ 93,000	$ 49,000	$ 48,000		$ 34,000	
Oxidation/ Coagulation Contactor	$ -						
MF System Upgrades	$ 288,000				$ 288,000		
Roughing Filter/ Mn & Fe Reduction System	$ 486,000			$ 486,000			
DBP Reduction System	$ 1,152,000	$ 50,000	$ 1,102,000				
TW Tank and Fairview PS	$ 191,000	$ 191,000					
Return Water System	$ 185,000			$ 185,000			
Chemical Systems	$ 186,000	$ 11,000		$ 33,000		$ 60,000	$ 82,000
Electrical and Instrumentation (17%)	$ 462,000	$ 59,000	$ 196,000	$ 128,000	$ 49,000	$ 16,000	$ 14,000
Subtotal	$ 3,174,000	$ 404,000	$ 1,347,000	$ 880,000	$ 337,000	$ 110,000	$ 96,000
Division 1 Costs (@ 8%)	$ 254,000	$ 32,000	$ 108,000	$ 70,000	$ 27,000	$ 9,000	$ 8,000
Taxes - Material Costs (@ 9.25%)	$ 321,000	$ 41,000	$ 136,000	$ 89,000	$ 34,000	$ 11,000	$ 10,000
Contractor OH&P (@ 10%)	$ 318,000	$ 40,000	$ 135,000	$ 88,000	$ 34,000	$ 11,000	$ 10,000
Estimate Contingency (@ 25%)	$ 794,000	$ 101,000	$ 337,000	$ 220,000	$ 84,000	$ 28,000	$ 24,000
Total Lessalt Bid Estimate	$ 4,861,000	$ 618,000	$ 2,063,000	$ 1,347,000	$ 516,000	$ 169,000	$ 148,000
Ridgemark PS Bid Estimate	$ 210,000	$ 210,000					
Design (@ 8%)	$ 389,000	$ 49,000	$ 165,000	$ 108,000	$ 41,000	$ 14,000	$ 12,000
Construction Mgmt (@ 10%)	$ 487,000	$ 62,000	$ 206,000	$ 135,000	$ 52,000	$ 17,000	$ 15,000
Subtotal Lessalt Project Estimate	$ 5,947,000	$ 939,000	$ 2,434,000	$ 1,590,000	$ 609,000	$ 200,000	$ 175,000
Property Cost @ $100K per acre	$ -						$ -
Total Lessalt Project Estimate	**$ 5,947,000**	**$ 939,000**	**$ 2,434,000**	**$ 1,590,000**	**$ 609,000**	**$ 200,000**	**$ 175,000**
Percent		**19%**	**49%**	**32%**	**12%**	**4%**	**3%**

Capital Costs: Alternative 3, Microfiltration + GAC

LESSALT WTP IMPROVEMENTS - DBP SOLUTION - for 2 MGD						
Cost Components	**TOTAL**	**Hydraulic Capacity, TW Tank, & Pump Stations**	**GAC System**	**Membrane Replacement - Fe & Mn Removal**	**Coagulation**	**Chemical Storage Building**
Sitework and Yard Piping	$ 186,000	$ 93,000	$ 49,000	$ 10,000	$ 34,000	
Oxidation/ Coagulation Contactor	$ 114,000				$ 114,000	
MF System Upgrades / Mn & Fe Reduction System	$ 445,000			$ 445,000		
DBP Reduction System	$ 1,152,000	$ 50,000	$ 1,102,000			
TW Tank and Fairview PS	$ 152,000	$ 152,000				
Return Water System	$ 57,000		$ 57,000			
Chemical Systems	$ 170,000	$ 11,000		$ 33,000	$ 60,000	$ 66,000
Electrical and Instrumentation (17%)	$ 386,000	$ 52,000	$ 205,000	$ 83,000	$ 35,000	$ 11,000
Subtotal	$ 2,662,000	$ 358,000	$ 1,413,000	$ 571,000	$ 243,000	$ 77,000
Division 1 Costs (@ 8%)	$ 213,000	$ 29,000	$ 113,000	$ 46,000	$ 19,000	$ 6,000
Taxes - Material Costs (@ 9.25%)	$ 318,000	$ 43,000	$ 169,000	$ 68,000	$ 29,000	$ 9,000
Contractor OH&P (@ 10%)	$ 266,000	$ 36,000	$ 141,000	$ 57,000	$ 24,000	$ 8,000
Estimate Contingency (@ 25%)	$ 666,000	$ 90,000	$ 353,000	$ 143,000	$ 61,000	$ 19,000
Total Lessalt Bid Estimate	$ 4,125,000	$ 556,000	$ 2,189,000	$ 885,000	$ 376,000	$ 119,000
Ridgemark PS Bid Estimate	$ 210,000	$ 210,000				
Design (@ 8%)	$ 330,000	$ 44,000	$ 175,000	$ 71,000	$ 30,000	$ 10,000
Construction Mgmt (@ 10%)	$ 414,000	$ 56,000	$ 219,000	$ 89,000	$ 38,000	$ 12,000
Subtotal Lessalt Project Estimate	$ 5,079,000	$ 866,000	$ 2,583,000	$ 1,045,000	$ 444,000	$ 141,000
Property Cost @ $100K per acre	$ -					
Total Lessalt Project Estimate	**$ 5,079,000**	**$ 866,000**	**$ 2,583,000**	**$ 1,045,000**	**$ 444,000**	**$ 141,000**

Percent		**21%**	**61%**	**25%**	**11%**	**3%**

O&M Costs (2 MGD): Alternative 1, Microfiltration + Nanofiltration

Design Flow 2 MGD

Summary	Cost ($/yr)
Power	$ 399,000
Incremental Raw Water Supply	$ 72,000
Chemicals	$ 144,000
MF & NF Membrane Replacement	$ 41,000
Laboratory Testing	$ 34,000
Disposal	$ 246,000
Maintenance	$ 20,000
Labor	$ 80,000
Misc. Administrative Costs	$ 14,000
Total O&M Cost/ Yr.	$ 1,050,000

Power Costs	HP	Number (Duty)	Total HP	kW	Hrs/ Yr	$/KWh	Cost ($/yr)
Influent pumping (9L)	29	2	58	43.25	8760	$0.17	$64,409
NF Feed Pumps	90	1	90	67.11	8760	$0.17	$99,945
NF CIP Pumps	75	1	75	55.93	416	$0.17	$3,955
MF Backwash Blowers	25	1	25	18.64	1402	$0.17	$4,442
Membrane Backwash Pump	25	1	25	18.64	1402	$0.17	$4,442
MF CIP Pumps	40	1	40	29.83	624	$0.17	$3,164
Distribution Transfer Pumps (Ridgemark)	43	2	86	64.13	8760	$0.17	$95,503
Fairview Booster Pumps	35	2	70	52.20	8760	$0.17	$77,735
Disposal Pumps	30	1	30	22.37	3903	$0.17	$24,986
Misc. Electrical							$20,000
Total Energy Costs				**372.10**			**$398,580**

Chemical Costs	lbs/day	lbs/yr	$/lb	Cost ($/yr)
Sodium Permanganate (0.5 mg/L)	9	3285	$6.44	$21,155
Antiscalant (2.5 mg/L)	42	15330	$1.47	$22,535
Filtration Polymer (<1 mg/L)	0	0	$1.25	$0
Caustic				$6,700
Disinfection Chemicals (2.5 mg/L chlorine)	42	15330	$1.50	$22,995
NF Cleaning Chemicals (1 duty)				$20,000
MF Cleaning Chemicals (2 duty)				$13,333
Coagulant, ACH (20 mg/L)	334	121910	$0.30	$36,573
Total Chemical Cost/ Year				$143,292

Membrane Replacement Costs	Unit Cost	Units	Total Cost	Cost ($/yr)
NF Membrane Replacement Cost (@ 7 years)	$900	60	$54,000	$7,714
MF Membrane Replacement Cost (@7 Years)	$800	288	$230,400	$32,914
Total Membrane Cost/Yr				$40,629

Disposal	Monthly Service Fee	Unit Price / HCF	HCF/Year	Cost ($/yr)
Sewage Disposal -MF (BW)	$ 75.45	$3.16	77425	$245,570

O&M Costs (2 MGD): Alternative 2, Greensand/ Anthracite + GAC + (Microfiltration)

Design Flow 2 MGD

Summary	With Membrane ($/yr)	Without Membranes ($/yr)
Power	$ 282,000	$ 263,000
Incremental Raw Water	$ 19,000	$ 19,000
Chemicals	$ 102,000	$ 88,000
MF Membrane Replacement	$ 42,000	$ -
GAC Replacement	$ 481,000	$ 481,000
Laboratory Testing	$ 34,000	$ 34,000
Disposal	$ 66,000	$ 66,000
Maintenance	$ 20,000	$ 14,000
Labor	$ 80,000	$ 54,000
Misc. Administrative Costs	$ 14,000	$ 10,000
Total O&M Cost/ Yr.	$ 1,140,000	$ 1,029,000

Power Costs	HP	Number (Duty)	Total HP	kW	Hrs/ Yr	$/KWh	Cost w/ Membranes ($/yr)	Cost w/o Membranes ($/yr)
Influent pumping (9L)	29	2	58	43.25	8760	$0.17	$64,409	$64,409
MF Backwash Blowers	25	1	25	18.64	1121	$0.17	$3,554	$0
Distribution Transfer Pumps (Ridgemark)	43	2	86	64.13	8760	$0.17	$95,503	$95,503
Fairview Booster Pumps	35	2	70	52.20	8760	$0.17	$77,735	$77,735
GAC/ Greensand Filter Backwash Pump	44	1	44	32.81	261	$0.17	$1,454	$1,454
Membrane Backwash Pump	25	1	25	18.64	1121	$0.17	$3,554	$0
MF CIP	40	1	40	29.83	499	$0.17	$2,531	$0
Disposal Pumps	45	1	45	33.56	568	$0.17	$12,398	$3,239
Misc. Electrical							$20,000	$20,000
Total Energy Costs				**293.06**			**$281,136**	**$262,339**

Chemical Costs	lbs/day	lbs/yr	$/lb	Cost w/ Membranes ($/yr)	Cost w/o Membranes ($/yr)
Sodium Permanganate (0.5 mg/L)	9	3285	$6.44	$21,155	$21,155
Sodium Bisulfite (optional)	9	3285	$0.18	$591	$0
Filtration Polymer (<1 mg/L)	0	0	$1.25	$0	$0
Caustic				$6,700	$6,700
Disinfection Chemicals (2.5 mg/L chlorine)	42	15330	$1.50	$22,995	$22,995
MF Cleaning Chemicals (2 duty+1 membrane)				$13,333	
Coagulant, ACH (20 mg/L)	334	121910	$0.30	$36,573	$36,573
Total Chemical Cost/ Year				$101,348	$87,423

MF Membrane Replacement Costs	Unit Cost	Units	Total Cost	Cost ($/ yr)
MF Membrane Replacement Cost (@7 Years)	$1,000	288	$288,000	$41,143

GAC Replacement Costs	tons/yr	$/ ton	Cost ($/yr)
GAC Replacement Cost	157.5	$3,050	$480,463

Disposal	Monthly Service Fee	Unit Price / HCF	HCF/Year	Cost ($/yr)
Sewage Disposal -Greensand BW	$ 76.45	$3.16	20495	$65,681

O&M Costs (2 MGD): Alternative 3, Microfiltration + GAC

Design Flow	2	MGD

Summary	Cost ($/yr)
Power	$ 281,000
Incremental Raw Water Cost	$ 48,000
Chemicals	$ 101,000
MF Membrane Replacement	$ 33,000
GAC Replacement	$ 481,000
Laboratory Testing	$ 34,000
Disposal	$ 164,000
Maintenance	$ 20,000
Labor	$ 80,000
Misc. Administrative Costs	$ 14,000
Total O&M Cost/ Yr.	$ 1,256,000

Power Costs	HP	Number (Duty)	Total HP	kW	Hrs/ Yr	$/KWh	Cost ($/yr)
Influent pumping (9L)	29	2	58	43.25	8760	$0.17	$64,409
MF Backwash Blowers	25	1	25	18.64	1402	$0.17	$4,442
Distribution Transfer Pumps (Ridgemark)	43	2	86	64.13	8760	$0.17	$95,503
Fairview Booster Pumps	35	2	70	52.20	8760	$0.17	$77,735
GAC Backwash Pumps	44	1	44	32.81	261	$0.17	$1,454
Membrane Backwash Pump	25	1	25	18.64	1402	$0.17	$4,442
MF CIP	40	1	40	29.83	624	$0.17	$3,164
Disposal Pumps	11	1	11	8.20	6570	$0.17	$9,162
Misc. Electrical							$20,000
Total Energy Costs				**267.71**			**$280,310**

Chemical Costs	lbs/day	lbs/yr	$/lb	Cost ($/yr)
Sodium Permanganate (0.5 mg/L)	9	3285	$6.44	$21,155
Filtration Polymer (<1 mg/L)	0	0	$1.25	$0
Caustic				$6,700
Disinfection Chemicals (2.5 mg/L chlorine)	42	15330	$1.50	$22,995
MF Cleaning Chemicals (2 duty+1 membrane)				$13,333
Coagulant, ACH (20 mg/L)	334	121910	$0.30	$36,573
Total Chemical Cost/ Year				$100,757

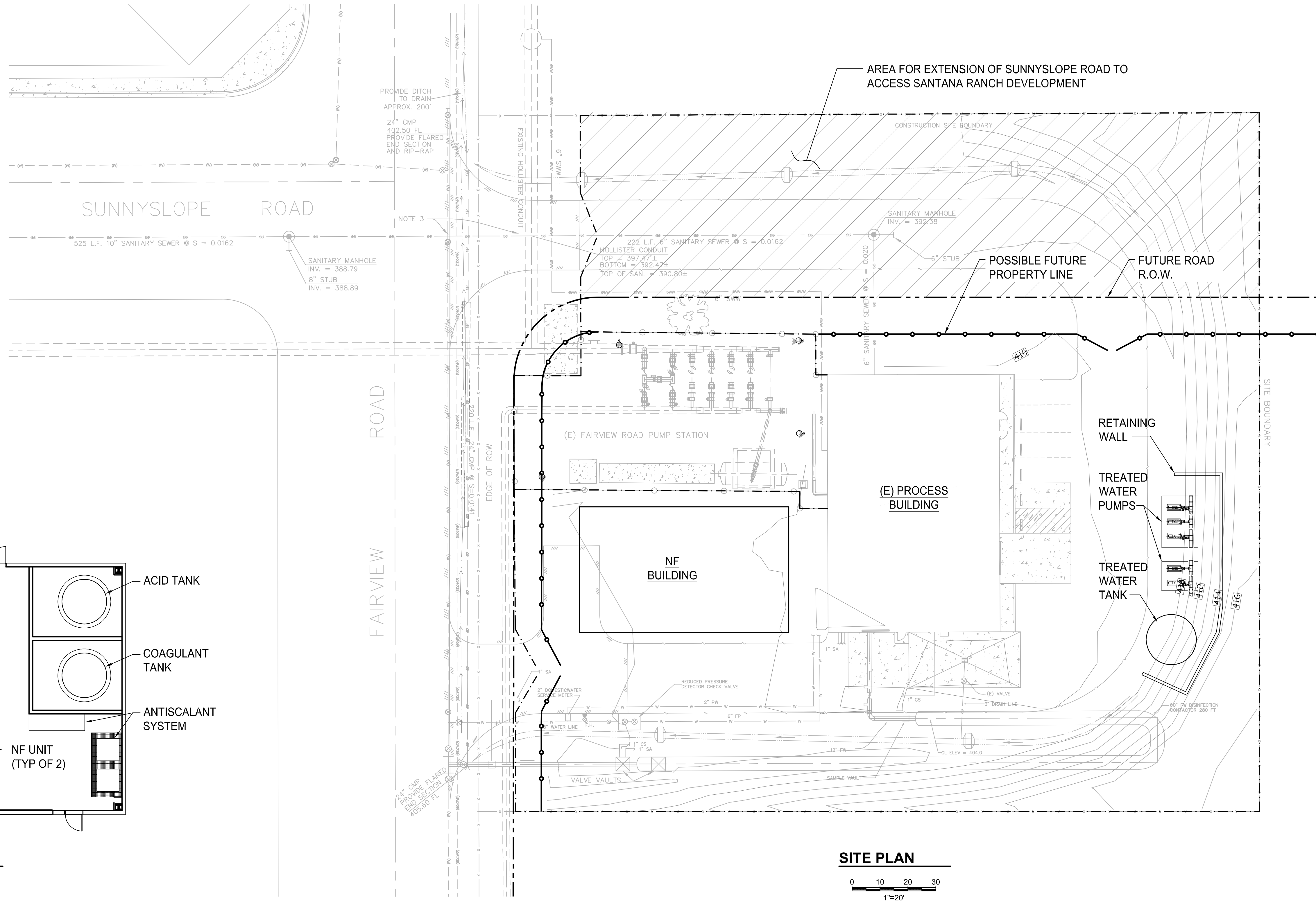
MF Membrane Replacement Costs	Unit Cost	Units	Total Cost	Cost ($/yr)
MF Membrane Replacement Cost (@7 Years)	$800	288	$230,400	$32,914

GAC Replacement Costs	tons/yr	$/ ton	Cost ($/yr)
GAC Replacement Cost	157.5	$3,050	$480,463

Disposal	Monthly Service Fee	Unit Price / HCF	HCF/Year	Cost ($/yr)
Sewage Disposal -MF backwash	$ 75.45	$3.16	51494	$163,626

APPENDIX B

Preliminary Site Plans



SITE PLAN

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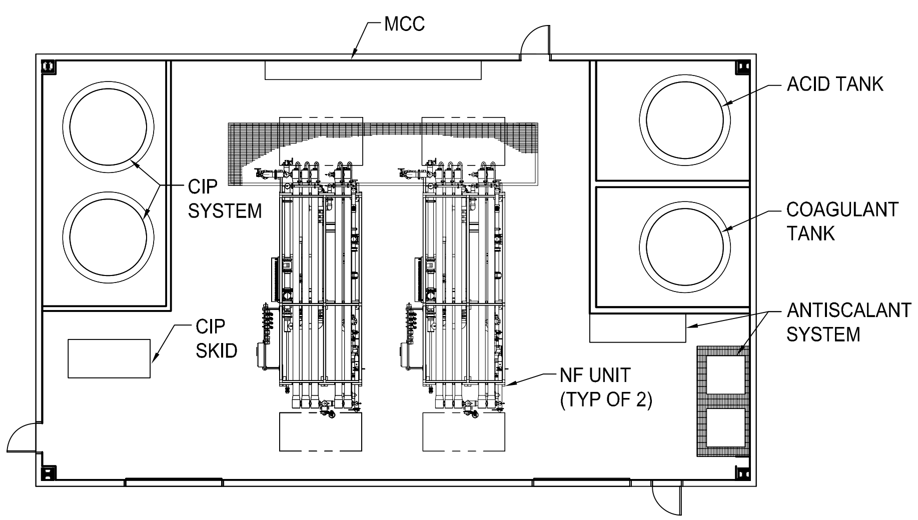

NF BUILDING PLAN

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USE OF DOCUMENTS	NO. / REVISION / DATE / BY	SCALES		DESIGNED ANK / DRAWN ANK / CHECKED TKR	SUNNYSLOPE COUNTY WATER DISTRICT	PRELIMINARY SITE PLAN	FILE NAME
THIS DOCUMENT, INCLUDING THE INCORPORATED DESIGNS, IS AN INSTRUMENT OF SERVICE FOR THIS PROJECT AND SHALL NOT BE USED FOR ANY OTHER PROJECT WITHOUT THE WRITTEN AUTHORIZATION OF KENNEDY/JENKS CONSULTANTS ©.		0 — 1" 0 — 25mm IF THIS BAR IS NOT DIMENSION SHOWN, ADJUST SCALES ACCORDINGLY.	REGISTERED PROFESSIONAL ENGINEER TODD K. REYNOLDS No. C59630 Exp. 12/31/11 CIVIL STATE OF CALIFORNIA		HOLLISTER, CALIFORNIA **LESSALT WTP DBP REDUCTION IMPROVEMENTS PROJECT** Kennedy/Jenks Consultants © 303 SECOND STREET, SUITE 300 SOUTH, SAN FRANCISCO, CA 94107	**2 MGD PHASED ALTERNATIVE 1: MF - NF**	JOB NO. 1068012.01 DATE SHEET OF

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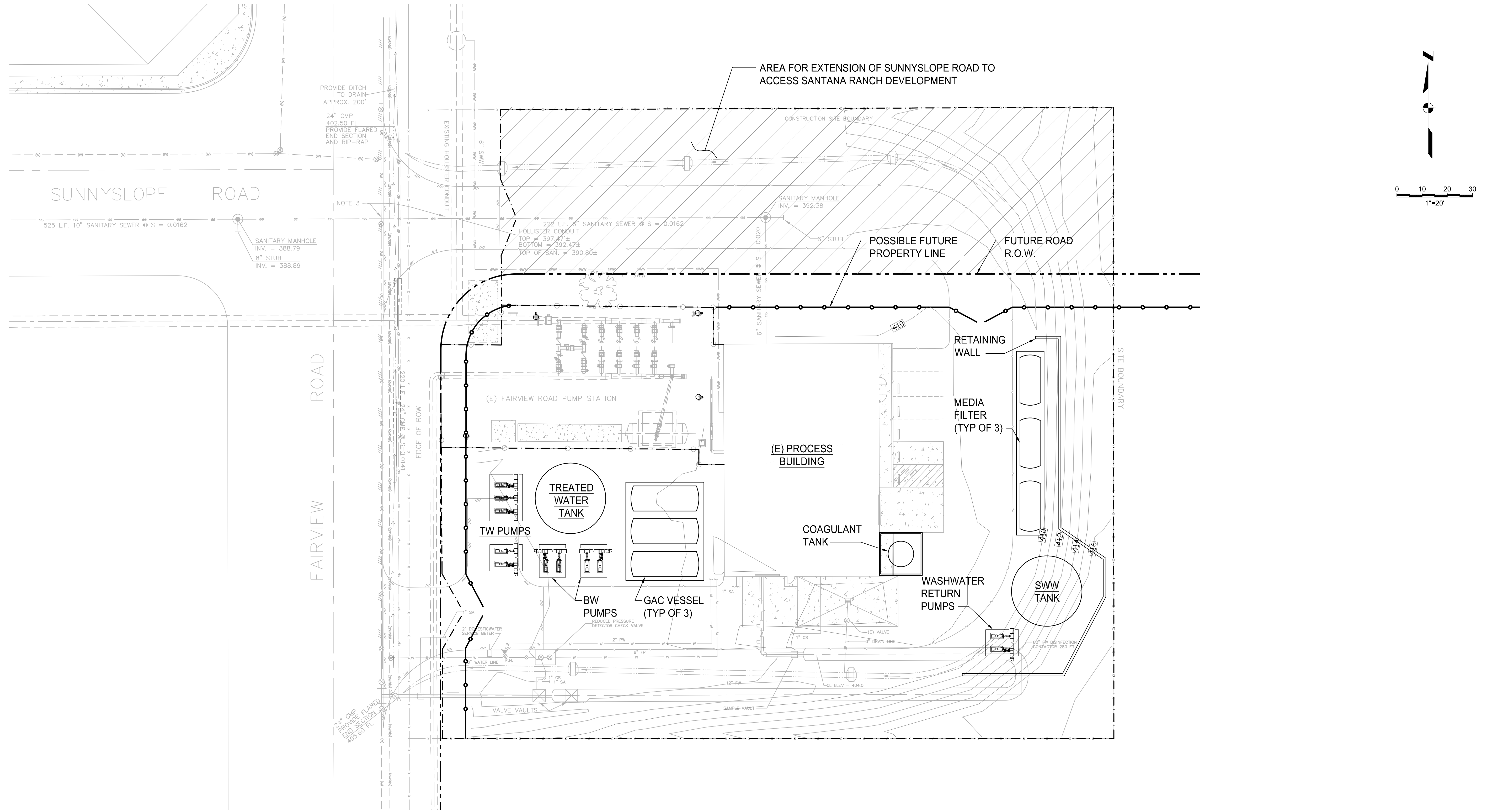

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Exp. 12/31/11
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STATE OF CALIFORNIA

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SUNNYSLOPE COUNTY WATER DISTRICT
HOLLISTER, CALIFORNIA

LESSALT WTP DBP REDUCTION IMPROVEMENTS PROJECT

Kennedy/Jenks Consultants ©
303 SECOND STREET, SUITE 300 SOUTH, SAN FRANCISCO, CA 94107

PRELIMINARY SITE PLAN
2 MGD PHASED ALTERNATIVE 2:
GREENSAND FILTER - GAC - MF

FILE NAME
JOB NO. 1068012.01
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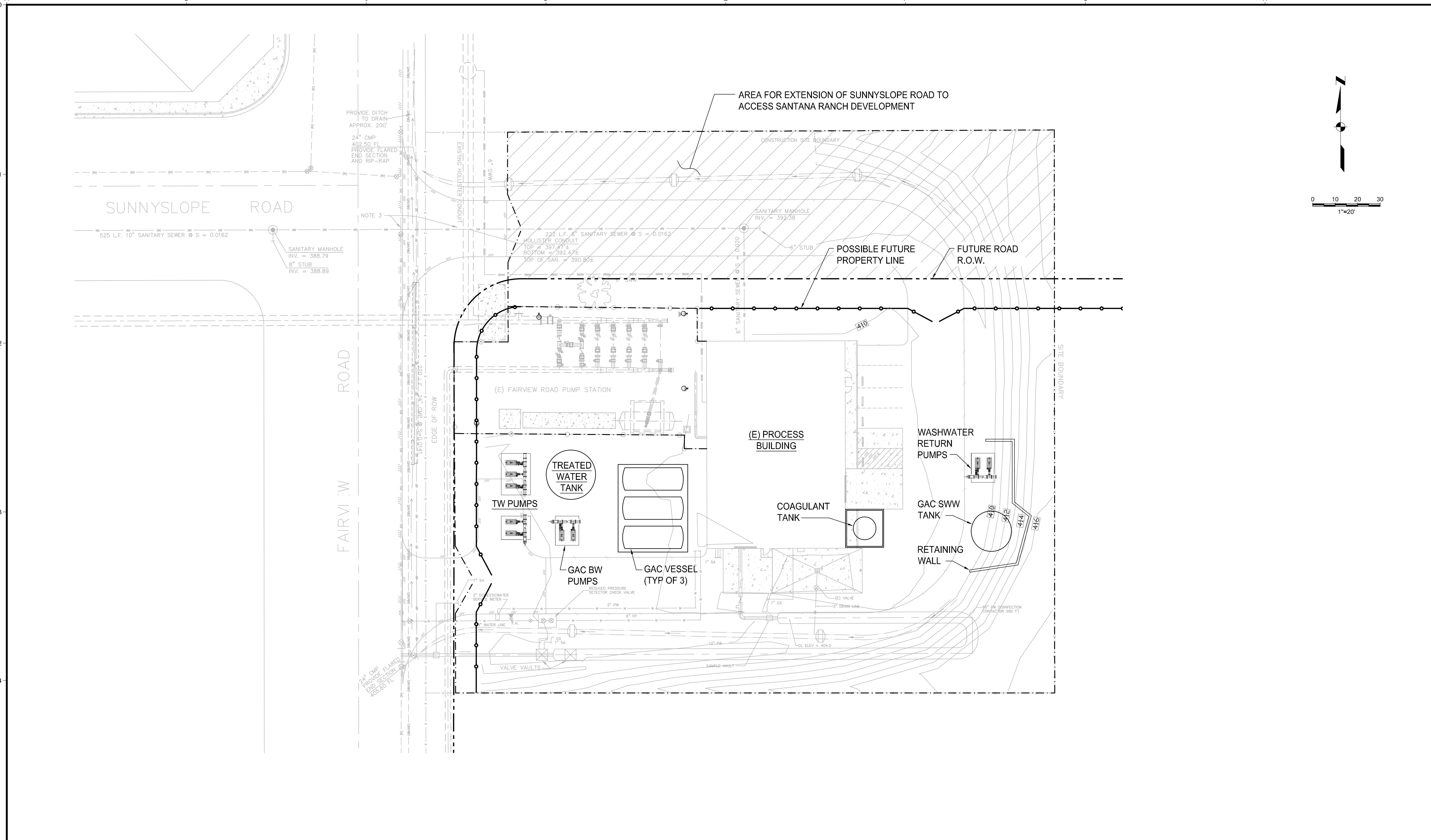

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DESIGNED ANK

DRAWN ANK

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SUNNYSLOPE COUNTY WATER DISTRICT

HOLLISTER, CALIFORNIA

LESSALT WTP DBP REDUCTION IMPROVEMENTS PROJECT

Kennedy/Jenks Consultants ©

303 SECOND STREET, SUITE 300 SOUTH, SAN FRANCISCO, CA 94107

PRELIMINARY SITE PLAN
2 MGD PHASED ALTERNATIVE 3:
MF - GAC

FILE NAME

JOB NO. 1068012.01

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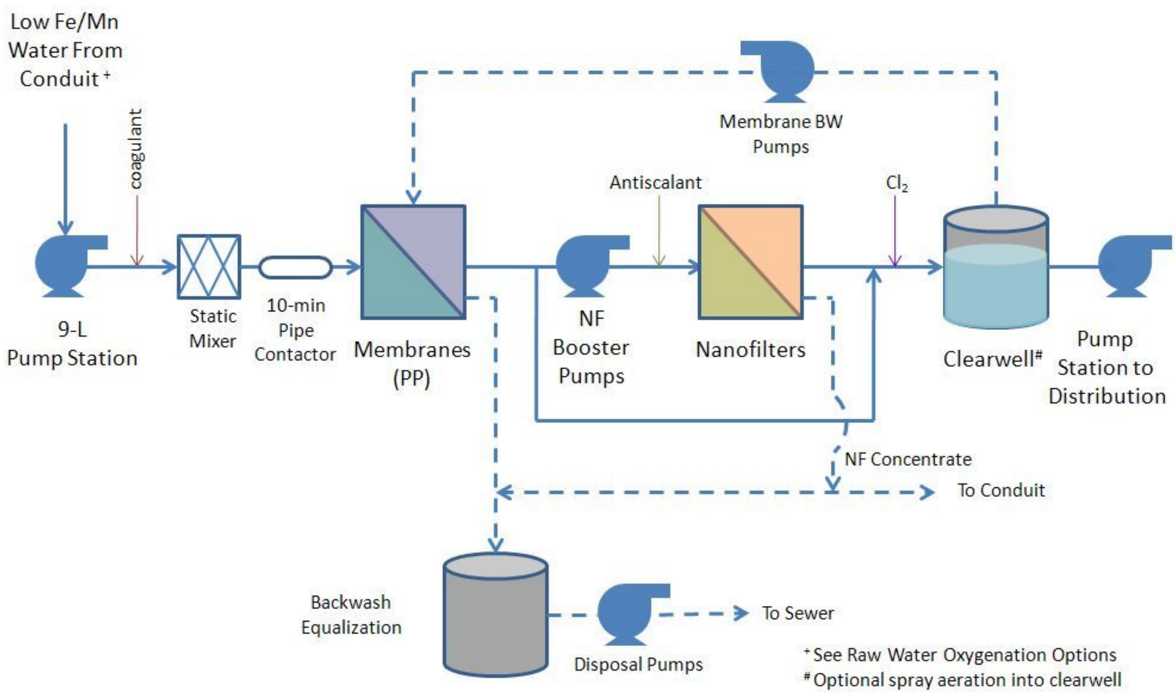
APPENDIX C

Source Water Options – Process Schematics and Costs

Source Water Options

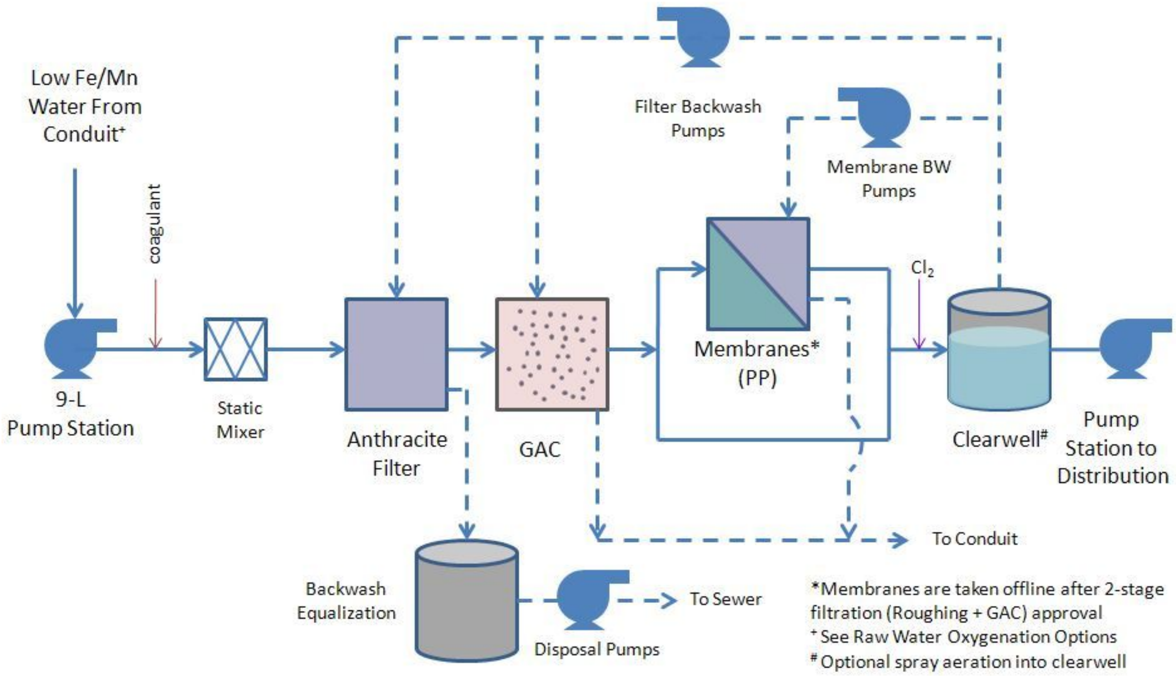
Iron and manganese mitigation can be attained using strategies that target the source water instead of relying on complete treatment at the Lessalt WTP. Conceptual process schematics shown below for Alternative 1A, 2A, and 3A modify the process alternatives to exclude Fe/ Mn treatment at the plant.

Alternative 1A – Microfiltration and Nanofiltration



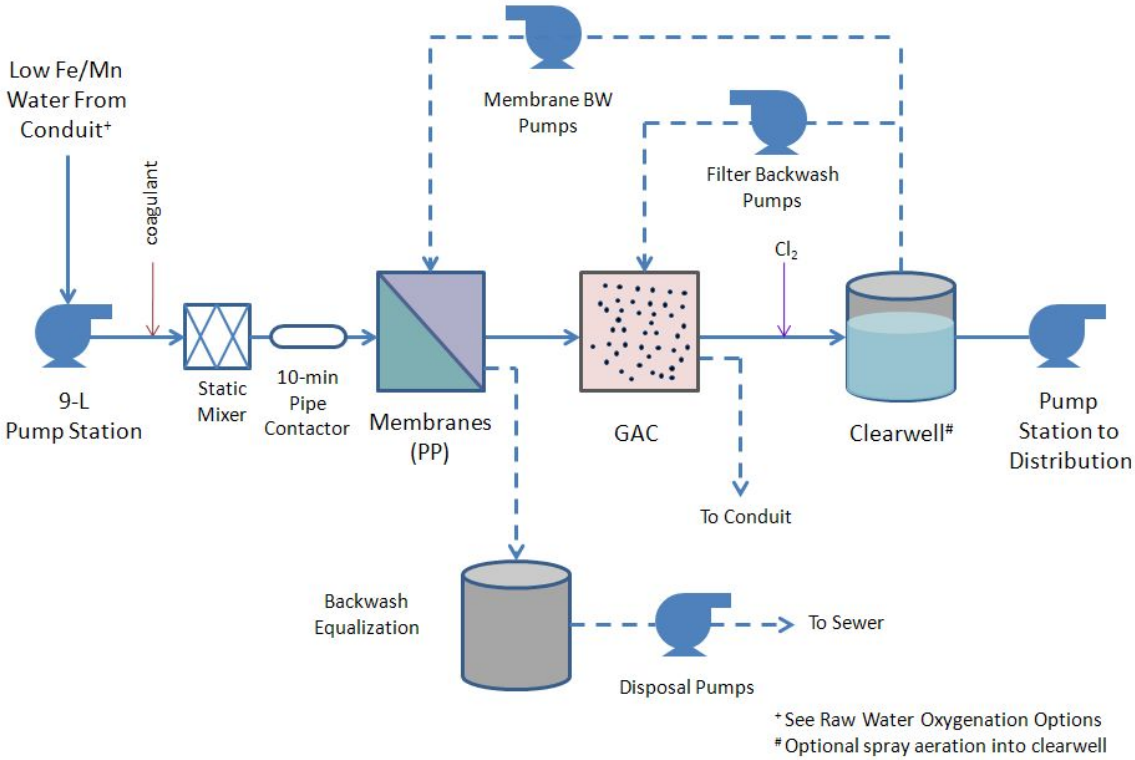
Alternative 1A is comprised of the same processes as Alternative 1 except for the chemical oxidation step. Alternative 1A also allows the plant to retain the PP microfiltration membranes and skids.

Alternative 2A - Greensand/ Anthracite Filtration, GAC, and Microfiltration



Alternative 2A differs from Alternative 2 by removing the chemical oxidation step and replacing greensand/ anthracite filters with anthracite/ sand filters.

Alternative 3A -Microfiltration and GAC



Alternative 3A differs from Alternative 3 by not requiring chemical oxidation and by retaining the existing PP membranes.

Source Water Costs

These process alternatives are only feasible if the influent source water concentrations of these constituents are low. The costs presented in Table C-1 shows the same three major process alternatives without iron and manganese oxidation or removal at the Lessalt WTP. These costs do not include the costs of reducing Fe/ Mn in the source water.

Table C-1. Lessalt WTP Process Alternatives Cost Comparison – No Fe/ Mn Treatment

Process Alternatives (No Fe/Mn Treatment at WTP)	WTP at 2 MGD Capacity			
	Capital Cost ($M)	Annual O&M Cost ($M)	Major MF Year 10 O&M Cost ($M)	Present Value Cost ($M)
1A – Microfiltration/ Nanofiltration (no WTP Fe/Mn Oxidation)	$6.04	$1.04	$0.58	$22.05
2A – Anthracite Filtration/ GAC/ (Microfiltration)	$5.33	Year 1-2: $1.12 Year 3-20: $1.01	N/A	$21.33
3A – Microfiltration/ GAC (no WTP Fe/Mn Oxidation)	$4.09	$1.24	$0.58	$23.17

As expected, removing the Fe/ Mn treatment processes from the alternatives reduces the overall capital, O&M, and present value costs at the plant. Alternative 2 still had the lowest total present value cost at $21.3 million, followed by Alternative 1 at $22.1 million. However, the cost differential between costs shown in Table 4 in the TM (costs with Fe/ Mn treatment) and in Table C-1 (costs without Fe/ Mn treatment) is smallest for Alternative 2. A major cost component for Alternatives 1 and 3 is the update of MF membranes to chemical-oxidant resistant PVDF. Foregoing chemical oxidation at the plant allows the existing MF membranes to remain in service and significantly reduces Alternative 1 and 3 costs.

If Fe/ Mn treatment at the Lessalt WTP is eliminated or decreased, the concentration of these constituents must be reduced in the source water by using one or more of the options described previously in the main TM document. Table C-2 shows the costs for these four previously mentioned options for reducing Fe/ Mn in the source water, i.e., reservoir mixing, reservoir oxygenation, pipeline oxygenation, or operational management. Although some of these costs were attractive in comparison to Fe/ Mn treatment at the plant, it was determined that none of the options were feasible for reasons described in the main TM document.

Table C-2. Lessalt WTP Source Water Options

Option	Source Water Option Description	WTP at 2 MGD Capacity Capital Cost ($M)	Annual O&M Cost ($M)	Present Value Cost ($M)
1	Reservoir Mixing	$0.72	$0.02	$1.03
2	Reservoir Oxygenation with Pure Liquid Oxygen	$0.76	$0.10	$2.28
	Reservoir Oxygenation with PSA[a] O_2 Generation System	$0.95	$0.07	$2.03
3	In-pipeline Oxygenation and Fe & Mn Removal at WTP[b]	varies	varies	varies
	with Alternative 1	$0.48	$0.02	$0.78
	with Alternative 2	$0.58	$0.03	$1.01
	with Alternative 3	$0.48	$0.02	$0.76
4	Operational Option	$0.24	$0.01	$0.31

Notes:

a. PSA = pressure swing adsorption. The PSA oxygen-generation system would be used in combination with liquid oxygen during high oxygen demand periods only, reducing overall liquid oxygen costs.

b. In-pipeline oxygenation options require removal of Fe and Mn at the Lessalt WTP using one of the three alternatives discussed previously. In-pipeline oxygenation replaces chemical (i.e. permanganate) oxidation.

EXHIBIT 7.10

WEST HILLS WATER TREATMENT PLANT

PRELIMINARY DESIGN REPORT, DECEMBER 2011

West Hills Water Treatment Plant Preliminary Design Report

City of Hollister • San Benito County • San Benito County Water District • Sunnyslope County Water District

HDR

West Hills Water Treatment Plant

Preliminary Design Report

City of Hollister

San Benito County

San Benito County Water District

Sunnyslope County Water District

December 30, 2011

REGISTERED PROFESSIONAL ENGINEER

Karen Pappas

NO. C77571

EXP. 1/7/13

STATE OF CALIFORNIA

Prepared under the responsible charge of

Karen Pappas
C77571

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Walnut Creek, CA 94596

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EXECUTIVE SUMMARY

This Preliminary Design Report (PDR) provides the basis of design for the new West Hills Water Treatment Plant (WHWTP) that will serve the Hollister Urban Area (HUA). This report establishes design criteria for each process and support system within the plant. Where design alternatives exist, this PDR describes the applicable advantages or disadvantages of each and presents a recommendation.

The result of this work is a preliminary design that meets or exceeds State and Federal regulatory requirements, achieves the source water specific water treatment goals, and contributes to the improvement of the overall HUA municipal water quality and reliability.

ES-1 Background

The addition of a new surface water treatment plant in the HUA meets the goals of the 2008 Hollister Urban Area Water and Wastewater Master Plan (Master Plan) regarding improved drinking water quality, improved reliability, and regional balance of water resources. In addition, the January 2010 Coordinated Water Supply and Treatment Plan (Coordinated Plan) updated the recommendations of the Master Plan and further describes the need for a new surface water treatment plant.

A Memorandum of Understanding (MOU) developed by the City of Hollister (City), San Benito County, the San Benito County Water District (SBCWD), and later the Sunnyslope County Water District (SSCWD) provided the initiative for both the Master Plan and Coordinated Plan. This group of agencies has together directed the preliminary planning and design effort related to the new WHWTP. Throughout this PDR, this group of agencies is referred to as the MOU Parties.

The WHWTP project, as described in the Coordinated Plan, was planned to provide:

- Approximately 6,000 acre-feet per year (AFY) of treated surface water in conjunction with the Lessalt Water Treatment Plant (LWTP), and
- A high quality drinking water supply to the western areas of the HUA that currently receive groundwater from municipal wells.

In August, 2010, HDR was retained by the MOU Parties to evaluate the source water quality, perform a treatment process screening evaluation, and later to develop the preliminary design for the WHWTP. The new treated surface water supply from the WHWTP will be blended with the existing high mineral content groundwater supply for overall improved water quality within the HUA distribution system. Water supply for the WHWTP will come from the San Luis and San Justo Reservoir via the Hollister Conduit.

ES-2 Objective

The primary objective of this project and the MOU Parties, as noted in the Coordinated Plan, is to provide high quality water supply to the western area of the City, an area which currently receives ground water.

In order to provide high quality treated water, the key treatment goals for the WHWTP include:

- Provide an initial maximum capacity of 6.0 million gallons per day (MGD) and a future capacity of 9.0 MGD.
- Reliably meet all applicable drinking water regulations, in particular the newly implemented Stage 2 Disinfectant/Disinfection Byproduct (DBP) Rule.
- Remove organic material, as measured by total organic carbon (TOC), from the source water such that any byproducts formed during disinfection within the 14 day distribution system water age remain within the regulated limits.
- Remove iron and manganese (which can have undesirable aesthetic impacts) from the source water

To address the project goals, the planning phase of this project included performing a detailed evaluation of the source water quality, as well as an alternatives analysis to select the preferred treatment system components. This report defines the recommended treatment approach to meet the project goals.

ES-3 Summary of Recommended Project

The following subsections describe the primary components of the project, as referenced in the Figure ES-1, and associated design recommendations.

ES-3.1 Process Description

The WHWTP process and facilities include a raw water pump station, raw and treated water transmission pipelines, pre-oxidation for iron and manganese removal, ballasted flocculation clarification pretreatment with enhanced organics removal, conventional gravity filtration, chemical feed and storage, treated water storage tank, and solids handling systems. Water will be pumped from the Hollister Conduit to the plant. Once on-site, the primary treatment processes, storage tank, and the distribution system will be fed by gravity.

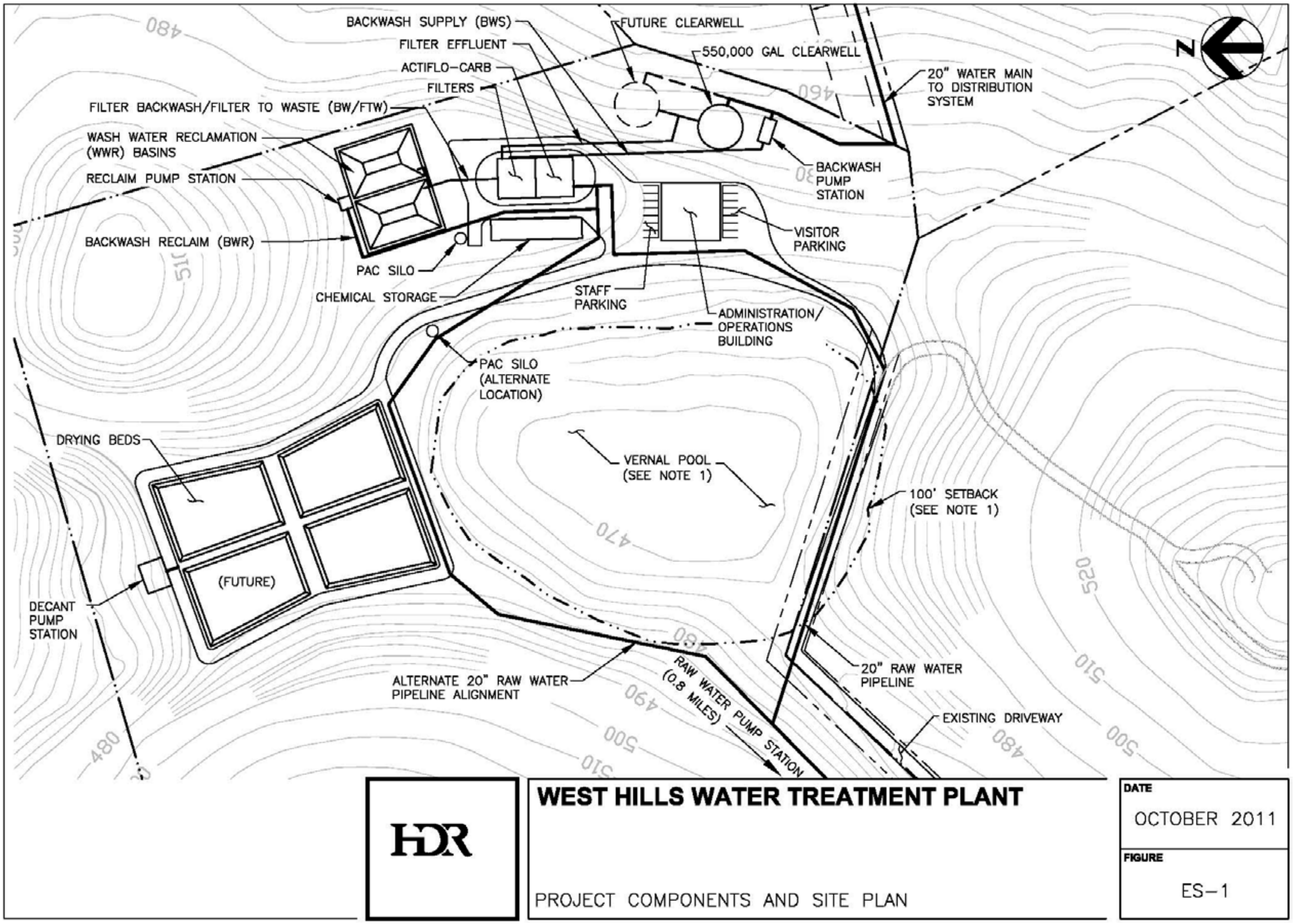

Figure ES 1. Site Plan

ES-3.2 Siting and Capacity Analysis

The West Area site, located in the hills to the north of Union Road, was selected for the future location of the WHWTP based on a cost and non-cost evaluation of five potential site alternatives in the HUA. The West Area site is currently jointly owned by the City and SSCWD.

The 6.0 MGD initial and 9.0 MGD build out capacities for the WHWTP are based on an assessment of both treatment and conveyance costs. Average annual capacity of the WHWTP is assumed to be 3.0 MGD (based on 6.0 MGD design capacity). Future average annual capacity of the WHWTP is assumed at 4.5 MGD (based on 9.0 MGD future design capacity).

ES-3.3 Regulatory Review

The regulatory review assessed the historical raw water quality data to determine the implications of the drinking water regulations on the design and operation of the new WHWTP and water distribution system. The WHWTP design is anticipated to provide the necessary tools to achieve and maintain compliance with the drinking water regulations. Consistent communication with the California Department of Public Health (CDPH) throughout the design phase will facilitate proactive regulatory compliance.

ES-3.4 Process Evaluation and Pilot Testing

During the planning stage of this project, a multi-stepped evaluation process was used to determine the preferred treatment strategy for the WHWTP. Four treatment technologies, each with advanced organics removal capabilities (leading to reduced DBP formation), were compared against specific screening criteria. Actiflo® Carb was selected over MIEX, GAC, and chloramination for use at the WHWTP. Pilot testing confirmed the treatment capabilities of Actiflo® Carb and established operating parameters for use in design.

ES-3.5 Raw Water Pump Station and Pipeline

The raw water pump station (RWPS) will be supplied from the Hollister Conduit, which transfers water from both the San Luis and San Justo Reservoirs. The RWPS will be located adjacent to the Hollister Conduit at the intersection of Richardson Road (the WHWTP access road) and Union Road, approximately 0.7 miles from the WTP site. Three 3.0 MGD pumps with variable frequency drives will maintain a target water level in the pretreatment (Actiflo® Carb) basins. Sodium permanganate will be stored and periodically dosed at the RWPS as a preoxidant for seasonal iron and manganese removal.

ES-3.6 Pretreatment

The Actiflo® Carb pretreatment system was selected from four treatment process alternatives for enhanced removal of organic material in the raw water. Pilot test data indicates that two 3.0 MGD Actiflo® Carb units will provide sufficient treatment to meet the Stage 2 DBP regulatory limits. The Actiflo® Carb system combines the standard Actiflo ballasted microsand clarification process with the recirculation of powder activated carbon (PAC) to enhance the removal (adsorption) of natural organic matter and taste and odor. Actiflo® Carb replaces conventional flocculation / sedimentation basins with a smaller footprint.

ES-3.7 Filtration

Downstream of the pretreatment, the filtration system at WHWTP will provide supplemental removal of turbidity, coagulated organic material, and oxidized particulate iron and manganese. Conventional gravity filtration was selected based on a review of three alternatives. For plants of this capacity, filters constructed of reinforced concrete are more cost effective than packaged metal systems. Three filters will be provided in the initial design and a fourth will be added during plant build out. Filter backwash will be supplied from the treated water clearwell.

ES-3.8 Chemical Feed and Storage

The chemical feed and storage facilities for the WHWTP are sized based on the anticipated average plant flow of 3.0 MGD (which corresponds to a design flow of 6.0 MGD). Where cost effective and appropriate for the overall design, the storage facilities are sized to store chemicals at the future average flow of 4.5 MGD. The chemical systems at the WHWTP site include sulfuric acid, PAC, polymer, coagulant (ferric chloride), sodium hydroxide, and sodium hypochlorite. In addition, sodium permanganate will be stored and fed at the RWPS site. The bulk chemical storage tanks will be located outdoors adjacent to the chemical feed pumps and systems, which will be covered by a metal canopy.

ES-3.9 Treated Water Storage and Pipeline

The proposed storage tank volume for the WHWTP includes sufficient storage capacity and disinfection contact time for the treated water prior to distribution. The sizing criteria for the storage tank includes two hours of storage at maximum day system water demand, filter backwash supply, on-site fire supply, and contact time (CT) disinfection requirements. The recommended storage for initial design is 550,000 gallon. Future build out to 9.0 MGD will include the addition of a second 550,000 gallon tank. The tank will be partially buried. Prestressed concrete (Type III construction) is recommended as the most economical alternative at the WHWTP.

ES-3.10 Solids and Backwash Handling

The recommended process for solids and backwash handling and treatment includes use of wash water reclamation (WWR) basins for storage and thickening of sludge followed by drying beds for supplemental drying. The two sources of solids that will require handling and disposal at the WHWTP include solids removed in the Actiflo® Carb pretreatment process and backwash streams from the filters.

Decant flow from the WWR basin will be returned to the head of the plant upstream of the pretreatment system. The thickened solids will be periodically emptied from the bottom of the thickener and transferred to the drying beds for further drying prior to disposal at the local landfill. Drying beds are recommended over mechanical dewatering due to lower capital and operating costs, simplicity of operation, and availability of site space.

ES-3.11 Plant Support Systems

Preliminary descriptions of plant support systems such as architectural, landscaping, site security, electrical and instrumentation systems, and site improvements are described for the WHWTP within this section. Solar and sustainability options are also included for consideration.

Architectural

The administration building will house office space, a control room, lab, multipurpose room, and workshop. The building exterior will have a ranch house residential style in keeping with the nearby neighborhood context. The WHWTP has the option to host solar panels through outright purchase or through a power purchase agreement (PPA) to provide a portion of the electricity needed to meet the demands of site equipment.

Landscaping

Landscaping within the facility will be kept to the structure and/or site perimeters to screen the facility, reduce water use, and minimize maintenance.

Site Security

Security measures to be included in the design of the WHWTP and RWPS focus on maintaining the facility's mission of providing safe and reliable drinking water to their customers. The recommended measures are designed to deter, detect, delay, and document undesired events, which include the damage or destruction of critical components and assets. Security recommendations include:

- Perimeter Fences and Gates
- Closed Circuit Video Cameras
- Intrusion Detection Alarms
- Landscaping
- Chemical Storage Security
- System Communication
- Chemical Storage Security

Electrical and Instrumentation

The electrical service and distribution system will be designed for the initial electrical load and the future plans to expand the plant to 9.0 MGD. The WHWTP requires two electrical services, the treatment plant and the RWPS located near the plant driveway entrance on Union Road. The total expected demand for the WTP is 230 kilo volt amperes (kVA). The total expected demand for the RWPS is 210 kVA. All equipment will be controlled from a central SCADA system to include vendor PLC's connected back to a central control room in or near the electrical room. Communication provisions will be installed along the plant pipeline to allow the RWPS to be controlled from the plant as well.

Civil Site Improvements

Civil site improvements include earthwork, site access and parking, signage, access roadway improvements for a minimum 24-foot wide road, site piping, sediment and erosion control, and fencing.

ES-4 Estimated Project Costs

The opinions of probable construction and operating costs for the WHWTP presented herein were developed from the preliminary design criteria described within this PDR, budgetary quotes from major equipment suppliers, standard cost estimating guidelines and engineering experience. Table ES-1 presents the preliminary opinion of probable construction and capital costs of the facilities for the WHWTP.

Table ES-1. Preliminary Opinion of Total Capital Cost for Water Treatment Plant Facilities

Facilities	Estimated Capital Cost
Raw Water Supply and Treated Water Pipelines, Bid Estimate	$4.210,000
Raw Water Pump Station, Bid Estimate	$1,015,000
West Hills Water Treatment Plant, Bid Estimate	$13,389,000
Total Construction Cost	$ 18,614,000
Design (at 8%) [(a)]	$1,489,000
Construction Management (at 10%)	$1,861,000
Property Acquisition	$100,000
Total Capital Coat	$22,064,000

Notes:
a) Design amount includes predesign costs.

The total probable construction cost is at $18,614,000. The total project capital cost (including engineering and construction services) is estimated at $22,064,000. Table ES-2 summarizes the preliminary opinion of probable project life cycle cost, based on the construction cost and present worth of the annual operating costs.

Table ES-2. Preliminary Opinion of Probable Project Life Cycle Cost

Cost Component	Estimated Cost
Capital	$ 22,064,000
Annual O&M	$ 1,273,000
Present Worth O&M[(a)]	$ 18,641,000
Total Life Cycle Cost	$ 42,283,000

Notes:
a) Present worth O&M cost assumes a 20 year period of analysis and a 3 percent discount rate.

ES-5 Project Schedule

A preliminary schedule for implementation of the final design, construction, and permitting plan is presented in Table ES-3. Based on a design notice to proceed in early January 2012, the design and bid phase would be completed in December 2012 with final testing and start up occurring in the first quarter of 2015.

Table ES-3. Preliminary Project Schedule

Milestone	Preliminary Schedule
Design Notice to Proceed	January 9, 2012
30 Percent Design Submittal	April 16, 2012
30 Percent Review	April 30, 2012
60 Percent Design Submittal	July 2, 2012
60 Percent Review	July 16, 2012
90 Percent Design Submittal	September 17, 2012
90 Percent Review	October 1, 2012
Advertise for Bids	October 29, 2012
Bid Period	October 30, 2012 - December 12, 2012
Construction	December 13, 2012 - January 8, 2015
Final Testing & Start-up	January 9, 2015 – March 30, 2015

Numerous federal, state, and local permits will be required for construction and operation of WHWTP. A comprehensive permitting action plan will be developed during the final design phase to minimize potential delays and mitigation costs. This strategy will include early contacts with critical regulatory agencies to define permitting needs.

ES-6 Next Steps

The following recommended next steps are critical for the implementation of the design of the WHWTP:

- Finalize the PDR based on review and comment by the MOU Parties.
- Complete recommended supplemental bench scale testing to simulate pre-oxidation and Actiflo® Carb pretreatment during high iron and manganese episodes in the 2011 fall season.
- Initiate the Final Design Phase of the project.
- Begin conducting monthly water quality sampling at a Hollister Conduit tap in the proximity of the proposed RWPS connection. This supplemental sampling will provide valuable information on the raw water quality within the Hollister Conduit that is most representative of future water supply to the WHWTP.
- Initiate CEQA and permitting.

1 INTRODUCTION

This Preliminary Design Report (PDR) provides the basis of design for the new West Hills Water Treatment Plant (WHWTP) that will serve the Hollister Urban Area (HUA). This report establishes preliminary design criteria for each process and support system within the plant. Where design alternatives exist, the sections within the PDR describe the applicable advantages or disadvantages of each alternative and present a recommendation.

The result of this work is a preliminary design that meets or exceeds state and national regulatory requirements, achieves the targeted water treatment goals, and contributes toward improvements to the overall HUA municipal water quality and reliability.

1.1 Project Background

The addition of a new surface water treatment plant in the HUA meets the goals of the 2008 Hollister Urban Area Water and Wastewater Master Plan regarding improved drinking water quality, improved reliability, and regional balance of water resources. In addition, the January 2010 Hollister Urban Area Master Plan Implementation Program Coordinated Water Supply and Treatment Plan (Coordinated Plan) updated the recommendations of the Master Plan and further describes the need for the new surface water treatment plant.

A Memorandum of Understanding (MOU) developed by the City of Hollister (City), San Benito County, the San Benito County Water District (SBCWD), and later the Sunnyslope County Water District (SSCWD) provided the initiative for both the Master Plan and Coordinated Plan. This group of agencies has together directed the preliminary planning and design effort related to the new WHWTP. Throughout this PDR, this group of agencies is referred to as the MOU Parties.

The WHWTP project, as described in the Coordinated Plan, was planned to provide:

- Approximately 6,000 acre feet (AF) of treated surface water in conjunction with the Lessalt Water Treatment Plant (LWTP), and
- A high quality drinking water supply to the western areas of the HUA that currently receive groundwater from municipal wells.

In August, 2010, HDR was retained by the MOU parties to evaluate the source water quality, perform a treatment process screening evaluation, and later to develop the preliminary design for the WHWTP.

1.1.1 Project Need and Description

The Master Plan and Coordinated Plan emphasized the goal of improving the quality of the municipal drinking water. The new treated surface water supply from the WHWTP will be blended with existing high mineral content groundwater supply for overall improved water quality within the HUA.

The WHWTP will be designed for an initial capacity of 6.0 million gallons per day (MGD) and a future buildout capacity of 9.0 MGD. The WHWTP process facilities include the raw water pump station, pre-oxidation for iron and manganese removal, raw water transmission pipeline, high rate pretreatment with enhanced organics removal, gravity filtration, chemical feed and storage, treated water storage, treated water transmission line, and solids handling and treatment. The raw water pumps lift the water from the Hollister Conduit to the plant site. Once on-site, the primary treatment processes and clearwell operate by gravity. Treated water from the clearwell flows by gravity into the distribution system near the corner of Nash Road and Line Street. Preliminary design criteria for each process are summarized in Appendix A. Preliminary design drawings that depict the process flow, hydraulic profile, site plan, and major project facilities, are included in Appendix B.

1.1.2 Existing Water System

The existing water system that serves the HUA consists of two interconnected systems that are operated by the City and SSCWD. Although the two agencies maintain independent service areas, their water supply and distribution systems are interconnected and can exchange water as necessary to satisfy customer demand. The HUA systems include a low, middle, and high pressure zone. Water supplies for the HUA are provided by groundwater and imported Central Valley Project (CVP) surface water supplies. Existing system facilities for water supply, treatment, and distribution include wells, the Lessalt WTP, pipelines, pump stations, and treated water storage reservoirs. Figure 1-1 shows the existing water system facilities for the HUA and the future site of the WHWTP.

The Lessalt WTP, a jointly-owned facility between the City and the SSCWD, was placed into operation in January 2003. The plant was designed to treat imported CVP water from the Hollister Conduit using microfiltration and chlorine disinfection. The treated water is distributed to both City and SSCWD customers.

The groundwater supply within the HUA is hard with elevated mineral content. The treated (lower mineral content) surface water from the new WHWTP will enter the southwest corner of the existing distribution system, through the low and middle pressure zones, and blend in the system with both surface water from the Lessalt WTP and groundwater.

1.1.3 Surface Water Supply

The surface water supply at the WHWTP originates from the Sacramento River-San Joaquin River Delta, is directed to San Luis Reservoir, and flows through the Hollister Conduit, as shown in Figure 1-1. Water conveyed from the San Luis Reservoir (SLR) to the Hollister Urban Area is diverted through the 1.8 mile long Pacheco Tunnel Reach 1 to the Pacheco Pumping Plant. The pumps initially lift the water which then flows by gravity through the Hollister Conduit to the HUA.

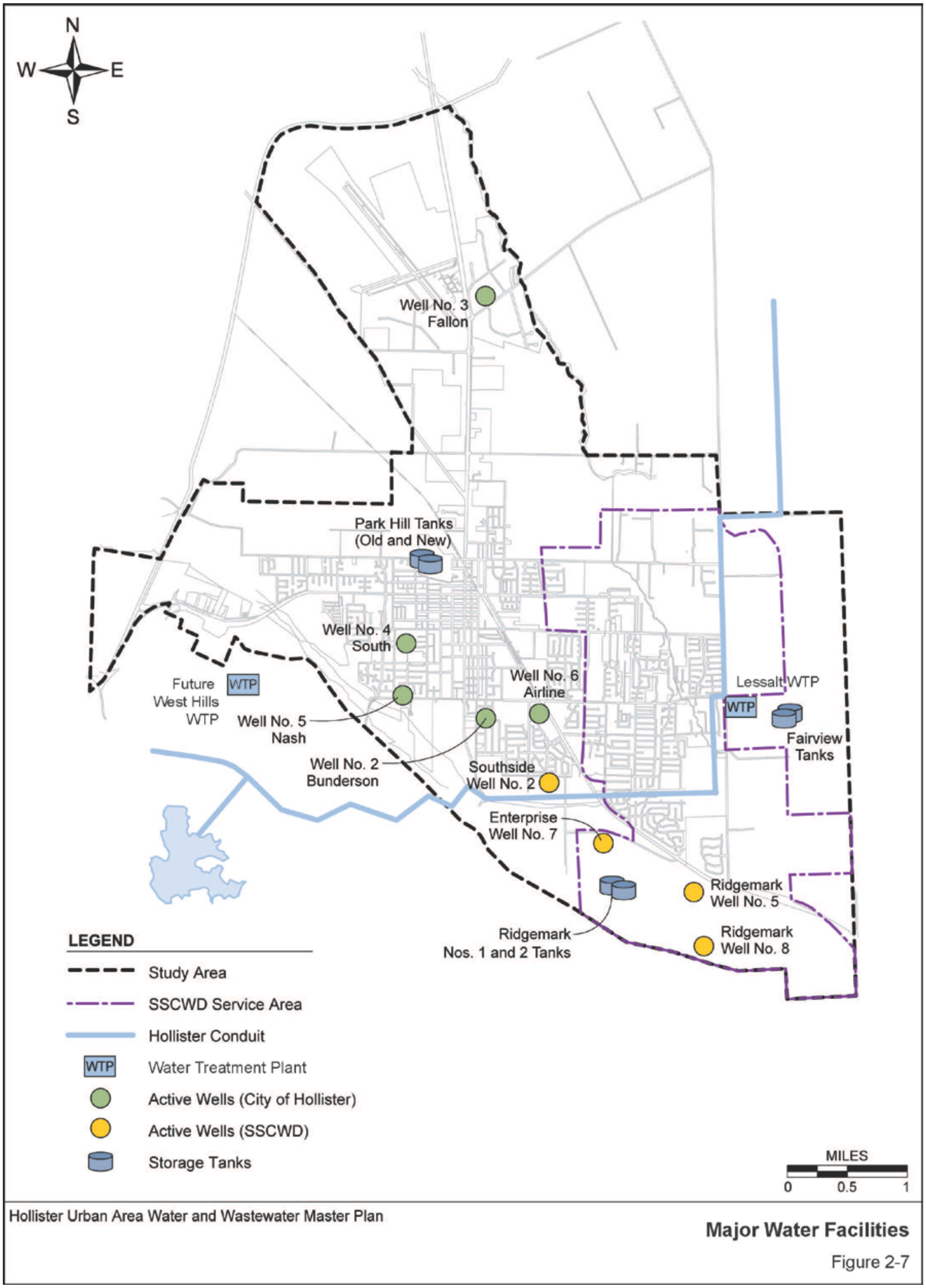

Figure 1-1. Hollister Urban Area Water System Facilities

The Hollister Conduit is a pressurized pipeline consisting of 60-inch and 42-inch diameter pipeline. It has a design capacity of 83 cfs and extends approximately 19.5 miles from the bifurcation with the Santa Clara Conduit to the terminus at San Justo Reservoir (SJR). The SJR has a storage capacity of 10,300 AF and is to the southwest of the City, as shown in Figure 1-1. Water supply for the WHWTP will come from the SLR and SJR via the Hollister Conduit.

1.1.4 Previous Studies

The completed studies that provided input to the development of this report include:

- *New Surface Water Treatment Plant Site Selection and Capacity Evaluation Technical Memorandum* (August, 2010), attached as Appendix C
- *Water Quality and Process Alternative Screening Technical Memorandum* (August, 2010) and *Process Update for New WTP* (November, 2010), attached as Appendix D
- *Actiflo® Carb Pilot Test Summary Technical Memorandum* (June, 2011), attached in Appendix E

1.2 Project Objectives

The primary objective of this project and the MOU Parties, as noted in the Coordinated Plan, is to provide high quality drinking water supply to the western area of the HUA, an area which currently receives groundwater.

In order to provide high quality treated water, the key treatment goals for the WHWTP include:

- Reliably meet all applicable drinking water regulations, in particular the newly implemented Stage 2 Disinfectant/Disinfection Byproduct (D/DBP) Rule.
- Remove organic material, as measured by total organic carbon (TOC), from the source water such that any byproducts formed during disinfection within the 14 day distribution system water age remain within the regulated limits.
- Remove iron and manganese (which can have undesirable aesthetic impacts) from the source water.

To address the project objectives, the planning phase of this project included a detailed evaluation of the source water quality, as well as an alternatives analysis to select the preferred treatment system components. The water quality analysis and treatment process selection is documented in the previously completed technical memorandums (TMs) listed in Section 1.1.4. This report defines the recommended treatment approach to meet the project objectives.

1.3 Report Organization

This report consists of individual sections, each of which describes the major components of the design, as listed in Table 1-1.

Table 1-1. Report Content

Section Number	Title
ES	Executive Summary
1	Introduction
2	Siting and Capacity Analysis
3	Regulatory Review
4	Process Evaluation
5	Raw Water Pump Station and Pipeline
6	Pretreatment (Actiflo® Carb System)
7	Filtration
8	Chemical Feed Systems
9	Treated Water Storage and Pipeline
10	Solids and Backwash Handling
11	Plant Support Systems
12	Design Codes, Studies, and Standards
13	Capital and O&M Cost Estimates
14	Project Schedule and Permitting
	Appendices

1.4 Abbreviations

To conserve space and improve the text, the following abbreviations have been used in this report:

AACE	Association for the Advancement of Cost Engineering
ac	acre
ACI	American Concrete Institute
ADA	Americans with Disabilities Act
ADD	average daily demand
AEIC	Association Edison Illuminating Companies
AF	acre-feet
AFY	acre-feet per year
AIHA	American Industrial Hygiene Association
AISC	American Institute of Steel Construction
ANSI	American National Standards Institute
AP	anodic protection
APCD	Air Pollution Control District
ASCE	American Society of Civil Engineers
ASHRAE	American Society of Heating, Refrigerating, and Air-Conditioning Engineers

ASME	American Society of Mechanical Engineers
ASTM	American Society for Testing and Materials
CAL/OSHA	State of California Occupational Safety and Health Administration
CARB	California Air Resources Board
CBC	California Building Code
CCAA	California Clean Air Act
CCR	Consumer Confidence Report
CDPH	California Department of Public Health.
CEC	California Energy Commission
CEQA	California Environmental Quality Act
CFE	combined filter effluent
cfs	cubic feet per second
City	City of Hollister
CMU	concrete masonry unit
Coordinated Plan	Coordinated Water Supply and Treatment Plan
County	San Benito County
CT	contact time
CVP	Central Valley Project
D/DBP	Disinfectant / Disinfection Byproduct
DBP	disinfection byproduct
Delta	Sacramento/San Joaquin Delta
DO	dissolved oxygen
DWR	California State Department of Water Resources
EIR	Environmental Impact Report
ENRCCI	Engineering News Record Construction Cost Index
FBRR	Filter Backwash Recycling Rule
FIRM	Flood Insurance Rate Maps
fps	feet per second
ft	feet
GAC	granular activated carbon
GMP	Groundwater Management Plan
gpm	gallons per minute
HAAs	haloacetic acids
HDLPE	High density linear polyethylene
HDXLPE	high density cross linked polyethylene

HGL	hydraulic grade line
hp	horsepower
HPC	heterotrophic plate count
HUA	Hollister Urban Area
ICEA	Insulated Cable Engineers
IDSE	initial distribution system evaluation
IEEE	Institute of Electrical and Electron Engineers
IESWTR	Interim Enhanced Surface Water Treatment Rule
IFE	individual filter effluent
IOCs	inorganic contaminants
IPCEA	Insulated Power Cable Engineers Association
ISA	Instrument Society of America
kVA	kilo volt amperes
LCR	Lead and Copper Rule
LRAAs	locational running annual averages
LT1ESWTR	Long Term 1 Enhanced Surface Water Treatment Rule
Lessalt WTP	Lessalt Water Treatment Plant
M&I	municipal and industrial
Master Plan	Hollister Urban Area Water and Wastewater Master Plan
MCLG	maximum contaminant level goal
MCLs	maximum contaminant levels
MF	microfiltration
MG	million gallons
MGD	million gallons per day
mg/L	milligrams per Liter
mL	milliliter
MOU	memorandum of understanding
MOU Parties	City of Hollister, San Benito County, San Benito County Water District, and Sunnyslope County Water District
MRDL	maximum residual disinfection level
MTBE	methyl tertiary-butyl ether
NAAMM	National Association of Architectural Metal Manufacturers
NEMA	National Electrical Manufacturers Association
NFPA	National Fire Protection Association
NPDES	National Pollutant Discharge Elimination System
NTU	nephelometric turbidity units

O&M	operation and maintenance
OSHA	Federal Occupational Safety and Health Administration
PAC	powder activated carbon
PDR	Preliminary Design Report
PG&E	Pacific Gas and Electric
PPA	Power Purchase Agreement
psi	pounds per square inch
PV	photovoltaic
PVC	polyvinyl chloride
RWPS	Raw Water Pump Station
RWQCB	California Regional Water Quality Control Board, Central Coast Region
SBCWD	San Benito County Water District
SCADA	supervisory control and data acquisition
SDS	simulated distribution system
SDWA	Safe Drinking Water Act
SIP	State Implementation Plan
SJR	San Justo Reservoir
SLR	San Luis Reservoir
SMACNA	Sheet Metal and Air Conditioning Contractors National Association
SOCs	synthetic organic chemicals
SSCWD	Sunnyslope County Water District
State	State of California
SWP	State Water Project
SWRCB	State Water Resources Control Board
SWTR	Surface Water Treatment Rule
TCR	Total Coliform Rule
TOC	total organic carbon
TDH	total design head
TDS	total dissolved solids
THMs	trihalomethanes
TM	Technical Memorandum
TSS	total suspended solids
TTHM Rule	Total Trihalomethane Rule
UCM	Unregulated Contaminant Monitoring
UL	Underwriters Laboratories, Inc.

USBR	United States Bureau of Reclamation.
USEPA	United States Environmental Protection Agency
VFD	variable frequency drives
VOCs	volatile organic compounds
WTP	water treatment plant
WHWTP	West Hills Water Treatment Plant
µg/L	micrograms per liter
yr	year

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2 Siting and Capacity Analysis

The results of the previously completed siting evaluation and treated water capacity analysis are detailed in the *New Surface WTP Site Selection and Capacity Evaluation Technical Memorandum* (August, 2010) included in Appendix C. The purpose of this section is to summarize the findings and recommendations of the evaluation and to describe updates or changes that have occurred since the completion of the TM, as well as their impact on the design assumptions.

2.1 Site Selection

The Coordinated Plan recommended that the WHWTP be located to provide high quality water to the western areas of the HUA which are currently served by groundwater wells with elevated hardness and total dissolved solids (TDS) concentrations. The site selection evaluation for the WHWTP included:

- Identification and evaluation of three locations for the new surface water treatment plant, based on discussions with the MOU Parties and past studies. The general locations include sites in the north, south, and west areas of the HUA as shown in Figure 2-1. Specific site locations maps are included in Appendix C.
- Site visits with photograph documentation to better understand site conditions.

2.1.1 Site Evaluation

A preliminary and final set of evaluation criteria were developed based on the principles and objectives of the MOU and to ensure consistency with other projects recommended within the Master Plan and Coordinated Plan. Input was also obtained from the Management Committee and the Water Supply and Treatment Subcommittee of the Governance Committee. The economic and non-economic criteria included:

- Minimize Cost
- Uniform Water Quality Distribution
- Raw Water Supply Reliability and Proximity
- Existing Land Use and Property Acquisition
- Flexibility for Future Expansion
- Compatibility with Future Demineralization project
- Plant Access and Serviceability
- Minimize Environmental Impacts

Table 2-1 summarizes the site evaluation, ranking, and weighting for each criterion.

Table 2-1. Summary of Site Evaluation

Alternative[a]	Minimize Costs	Uniform Water Quality Distribution[b]	Raw Water Supply Reliability and Proximity	Existing Land Use and Property Acquisition[c]	Subtotal	Flexibility for Future Expansion[d]	Future Demin Project Compatibility	Plant Access and Serviceability	Minimize Environ. Impacts	Subtotal	Total	Rank
	Primary Evaluation Criteria					Secondary Evaluation Criteria						
Evaluation Results												
North Site 1	1.0	5	1	1	8.0	5	3	5	3	16.0	24.0	2
North Site 2	1.0	5	1	1	8.0	5	3	5	3	16.0	24.0	2
South Site 1	1.0	5	5	1	12.0	5	5	1	1	12.0	24.0	2
South Site 2	1.0	5	5	1	12.0	5	5	1	1	12.0	24.0	2
West Site 1	5.0	5	3	5	18.0	5	5	3	3	16.0	34.0	1
Weighting	30%	15%	15%	15%	75%	10%	5%	5%	5%	25%	100%	
Weighted Evaluation Results												
North Site 1	3.0	8	2	2	13.5	5	1.5	2.5	1.5	10.5	24.0	4
North Site 2	3.0	8	2	2	13.5	5	1.5	2.5	1.5	10.5	24.0	4
South Site 1	3.0	8	8	2	19.5	5	2.5	0.5	0.5	8.5	28.0	2
South Site 2	3.0	8	8	2	19.5	5	2.5	0.5	0.5	8.5	28.0	2
West Site	15.0	8	5	8	34.5	5	2.5	1.5	1.5	10.5	45.0	1

Notes:

a) The numerical ranking is High = 5, Medium = 3, Low = 1.

b) North Sites connect directly to the Park Hill storage tanks.

c) North Sites are zoned for agricultural or rural residential; South and West Sites are zoned agricultural. South and North Sites are designated Prime Farmland. The west site parcels are owned by the City and SSCWD.

d) Specific parcels for the North Sites have not been identified, although several large parcels are currently for sale.

2.1.2 West Site Recommendation

Based on the site evaluation, the West Site best meets the evaluation criteria and is preferred over the other sites. The West Site is located in the hills to the north of Union Road as shown in Figures B-4 and B-5. Specific characteristics and advantages of this site include:

- Two adjacent parcels are jointly owned by the City and SSCWD.
- Site elevation facilitates gravity flow from the WHWTP to the distribution system, thereby reducing pumping requirements, improving reliability, and reducing energy costs.
- Accessible from Union Road.
- Total area is 32 acres; however, a previous environmental impact report found a vernal pool located on the northeast parcel, which could reduce the usable area to approximately 30 acres.
- Requires new connection to the Hollister Conduit and conveyance to the plant site.
- Requires new transmission pipeline from the plant to the distribution system near the intersection of Nash Road and Line Street.

2.1.3 Updates to the August 2010 Site Selection

Design related updates to the WHWTP site assumptions described in the *New Surface WTP Site Selection and Capacity Evaluation Technical Memorandum* (August 2010) include:

- Revisions to the treated water pipeline alignment. The pipe will exit from the southern end of the site and follow a route due east to Riverside Road. The revised alignment is shown in Figure B-5.
- Additional easements. Supplemental easements are required for approximately 300 ft of the revised treated water pipeline alignment.
- Access to the site. The preferred access to the site is by improvements made to the existing driveway which currently overlaps the southern buffer zone to the previously defined vernal pool area. The preliminary design assumes that the potential environmental impacts from the site access road can be mitigated. The current update to the site EIR is expected to address and confirm this assumption.

2.2 Capacity Analysis

As described in the January 2010 Coordinated Plan, sufficient surface water supply is available from a combination of existing CVP entitlements, North County groundwater, and additional imported surface water to provide water to both the Lessalt WTP and a second surface water treatment plant (WHWTP) within the HUA.

The planned capacity distribution between the existing Lessalt WTP and the new WHWTP was assessed based on treatment and conveyance project costs, in order to reduce overall costs while improving water quality throughout the distribution system. Three capacity distribution options were evaluated in the *New Surface WTP Site Selection and Capacity Evaluation Technical Memorandum* (August 2010) including:

1) Both Lessalt WTP and new WHWTP at 3 MGD,

2) Lessalt WTP at 1.5 MGD and new WHWTP at 4.5 MGD, and

3) Lessalt WTP at 0 MGD and new WHWTP at 6 MGD.

Option 2 was eliminated from further evaluation based on the results of the process alternatives screening. The recommended treatment process associated with the 1.5 MGD Lessalt WTP capacity option was removed from the process evaluation. Therefore, only capacity Options 1 and 3 were evaluated.

2.2.1 Capacity Evaluation and Recommendation

To compare the recommended capacity options, the following cost and non-cost factors were evaluated:

- Cost. Project costs were less than five percent different for Options 1 and 3.
- Reliability. Option 1 is preferred because two plants provide redundancy, less pumping, and shorter transmission pipelines.
- Uniform Water Quality. Option 1 is preferred because treated surface water is supplied from both the east and west sides of the distribution system.
- Asset Utilization. Option 1 is preferred to fully realize the prior capital investment and significant remaining useful life of the Lessalt WTP.
- Environmental Impact. Option 3 would require additional energy use and greater community impacts for construction of larger transmission pipelines. Therefore, Option 1 is preferred with respect to environmental impacts.

Based on the findings presented above, the recommendations from the *New Surface WTP Site Selection and Capacity Evaluation* (August 2010) were to:

- Complete predesign of the Lessalt WTP and size the plant for approximately 3 MGD.
- Initiate predesign for the new WHWTP at the West Site and size the plant for a capacity of approximately 3 MGD. With future expansion to 9.0 MGD or more depending on results of further analyses.

2.2.2 Updates to the August 2010 Capacity Evaluation

Design related updates to the WHWTP capacity assumptions that were described in the *New Surface WTP Site Selection and Capacity Evaluation Technical Memorandum*, (August 2010) include:

- Initial capacity design of the WHWTP revised to 6.0 MGD (from 3.0 MGD), including future expansion to 9.0 MGD.
- Average annual capacity of the WHWTP assumed at 3.0 MGD (based on 6.0 MGD design capacity). Future average annual capacity of the WHWTP is assumed at 4.5 MGD (based on 9.0 MGD future design capacity).

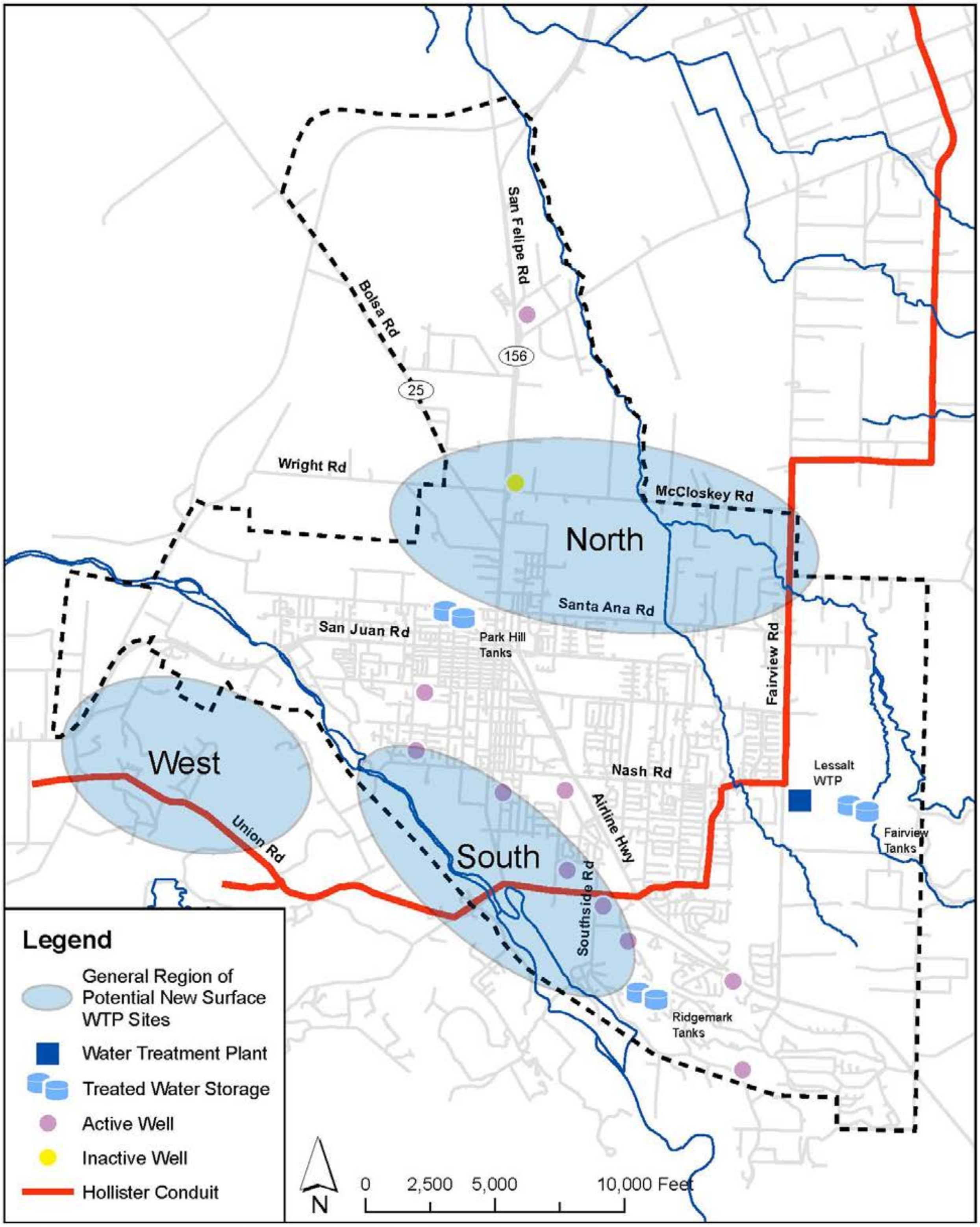

Figure 2-1. HUA site locations.

3 REGULATORY REVIEW

The objective of this Section is to describe the implications of the drinking water regulations on the design and operation of the new WHWTP and water distribution system. The level of treatment provided to meet the regulatory requirements, as described in the balance of this report, is based on the historical raw water quality data from the Lessalt WTP and limited additional sampling data points presented herein.

3.1 Background

The WHWTP will be supplied with water from the Hollister Conduit, which is a large diameter pipeline that conveys CVP water from SLR to San Benito County, where it is used for irrigation and municipal and industrial (M&I) purposes or stored in the SJR, a terminal storage reservoir. The historic operational strategy of the Hollister Conduit and associated reservoirs indicates that, during most of the year, the primary source of water supply will be from the SLR. However, during peak agriculture use periods the Hollister Conduit is back fed from the SJR, and thus the source water quality to the WHWTP will change. Additionally, future conduit / reservoir operations may include expanded use of SJR during the spring season, thus increasing the annual SJR supply to the plant. Treated water from the WHWTP will be fed into the middle pressure zone of the existing distribution system for the City, which is interconnected with the distribution system for SSCWD.

Water quality data was collected at the Lessalt WTP and is representative of the SLR supply. Due to current treatment limitations at the Lessalt WTP, when the plant begins to receive San Justo water in the fall, the operators shut it down, and as a result, no sampling has historically been conducted. The Lessalt WTP uses the processes of microfiltration and free-chlorine disinfection and faces challenges to reliably meet the Stage 2 D/DBP Rule. The design of treatment process upgrades at the Lessalt WTP (concurrent with design of the WHWTP) the will address those challenges.

The Lessalt WTP is owned by the Hollister – Sunnyslope Water Treatment Agency and is operated by the SSCWD. Treated water from the Lessalt WTP is sold wholesale to both the City of Hollister and SSCWD. The regulatory discussion in this Section assumes that similar to the Lessalt WTP, the WHWTP will also provide wholesale treated water to the City of Hollister and SSCWD.

Figure 3-1 shows the location of the water quality sampling site Lessalt WTP, the two water supply reservoirs, the Hollister Conduit, and the future location of the WHWTP.

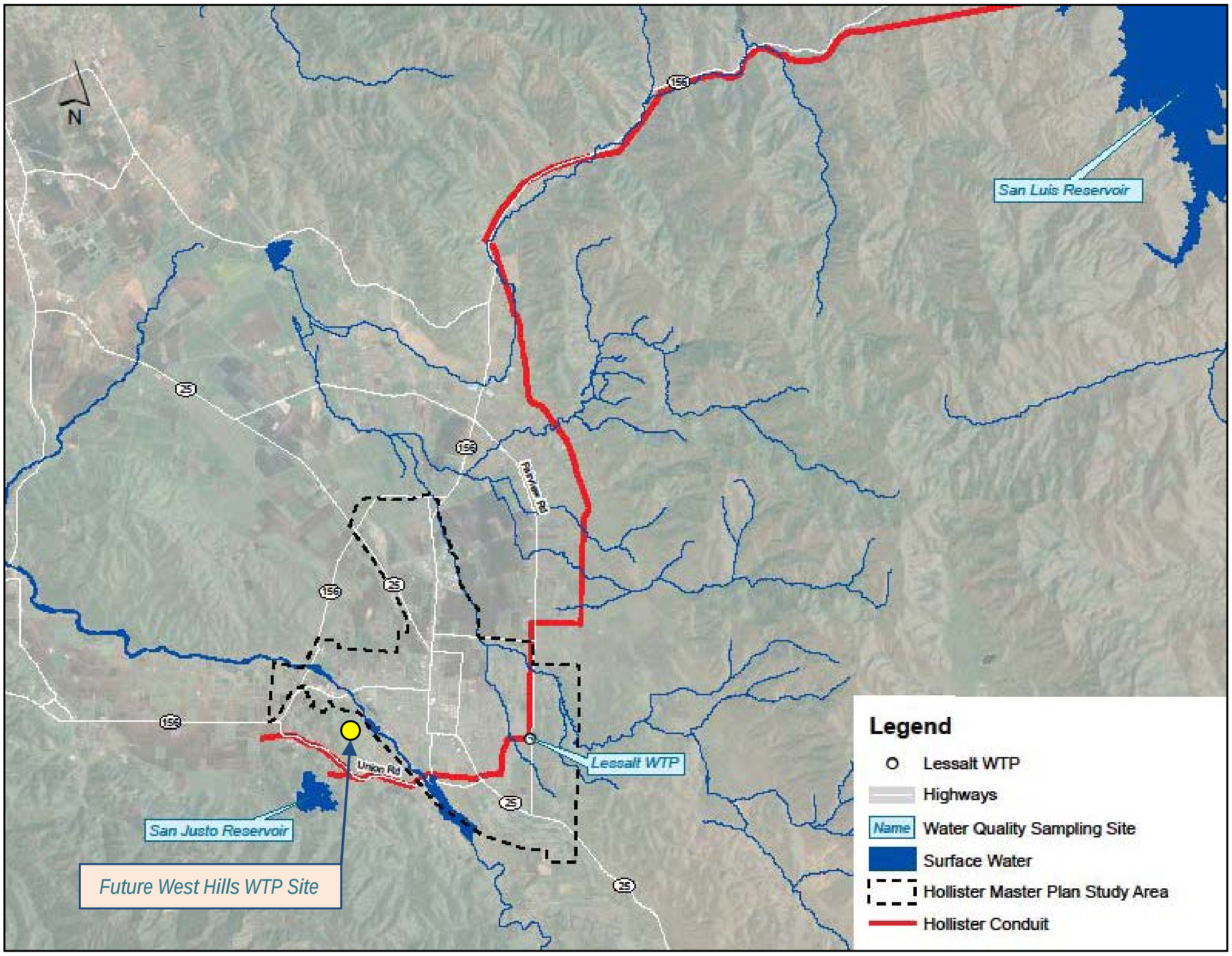

Figure 3-1. Water Quality Sampling Site, Water Supply Reservoirs, WTP Sites

3.2 Water Quality Data

The water quality data from weekly sample events at the Lessalt WTP (representing SLR water) and from independent sample events at the SJR are summarized in Table 3-1. The data represents the general anticipated quality of water to be treated at the WHWTP.

The SLR water is best characterized by moderate TOC, low turbidity, high bromide and seasonally high manganese levels. Although the TOC levels are moderate, the DBP formation potential is high, which is expected for CVP water. Water from the SJR is typically higher is TOC and contains elevated concentrations of iron and manganese which dramatically increase during the fall season when the dissolved oxygen level in the reservoir drops. There have been no positive tests for Cryptosporidium and the Lessalt WTP has a Bin Classification of Bin 1.

Table 3-1. Source Water Quality

Parameter	Units	Lessalt WTP (Representative of San Luis Reservoir)								San Justo Reservoir		Pilot Test (Feb – March, 2011)		
		2008-2009				2005-2007				Sampling Events		Phase I	SLR	
		Average	Min	Max	95%	Average	Min	Max	95%	March 23, 2010	October 13, 2010	Average	Average	Average
Total Coliform	MPN/100mL	273.5	<1	>2419.6	1208.0	185.8	<1	>2419.6	1986.3	-	-	-	-	-
Fecal Coliform	MPN/100mL	2.2	<1	8.6	7.27	3.7	<1	17.8	9.9	-	-	-	-	-
Cryptosporidium(a)	Oocysts/L	0	0	0	0	-	-	-	-	-	-	-	-	-
Giardia(b)	Cysts/L	0	0	0	0	-	-	-	-	-	-	-	-	-
E. Coli	MPN/100mL	2.02	0	4.1	3.6	-	-	-	-	-	-	-	-	-
Turbidity	NTU	3.1	0.81	8.98	5.76	2.0	0.21	6	4.44	1.3	11	4.9(c)	4.1(c)	3.1(c)
Total Alkalinity as $CaCO_3$	mg/L	86.9	70	130	110	83.8	58	150	110	110	94	69(d)	86(d)	75(d)
Bicarbonate as $CaCO_3$	mg/L	86.1	70	130	110	82.2	58	150	110	110	94	-	-	-
Bromide	mg/L	0.31	0.27	0.35	0.43	0.25	0.086	0.36	0.32	0.33	0.3	-	-	-
Calcium	mg/L	23.0	20	28	27	21.6	18	27	24	23	24	-	-	-
Carbonate as $CaCO_3$	mg/L	4.5	ND	12	8.2	5.46	1.5	12	10.6	ND	ND	-	-	-
Color	Units	15.7	0	45	25	15.2	9	100	22	15	50	-	-	-
Hardness	mg/L	117.5	100	150	140	105.5	93	140	130	130	120	98(d)	120(d)	100(d)
Iron	mg/L	0.07	0	0.3	0.12	0.07	0.02	0.42	0.12	0.073	1.7	0.33(d)	0.13(d)	0.19(d)
Magnesium	mg/L	14.8	13	19	18	12.9	11	18	15.5	17	15	-	-	-
Manganese	mg/L	0.07	0	0.43	0.24	0.06	0.01	0.68	0.15	0.15	0.25	7.0(d)	13.0(d)	4.7(d)
pH	-	8.1	6.5	8.8	8.4	8.0	7.37	8.76	8.4	8.1	8.2	7.5(c)	8.1(c)	7.8(d)
Total Dissolved Solids (TDS)	mg/L	-	-	-	-	302.5	250	350	330	360	320	-	-	-
TOC	mg/L	3.0	2.3	3.8	3.535	3.3	2.3	5	4.52	-	3.7	3.68(c)	4.24(c)	3.44(c)
DOC	mg/L	2.9	2.4	3.8	3.34	2.9	2.3	5.2	4.5	3.5	3.4	4.0(d)	4.3(d)	3.5(d)
UV 254	cm^{-1}	0.071	0.05	0.086	0.086	0.088	0.051	0.18	0.1474	0.071	0.14	0.11(c)	0.074(c)	0.10(d)

Notes:
(a) Sampled monthly.
(b) Giardia data is from (1) sample in 2008.
(c) Daily Online and Grab Samples.
(d) Five weekly grab samples

It is recommended that more extensive raw water quality data be collected from the San Justo Reservoir to better characterize the water quality throughout the design and construction process. The following standard water quality constituents should be monitored on a monthly basis: iron, manganese, TOC, DOC, UV254, pH, bromide, turbidity, color, odor, hardness, alkalinity and TDS. Additional routine sampling for other relevant constituents (i.e. microbial, metals, nitrates, etc.) shall be performed per applicable regulations. The sample location should be carefully chosen to best represent the source water. A suggested location representative of the future combined water supply to the WHWTP is at a Hollister Conduit sample tap near the planned connection for the RWPS.

3.3 Regulatory Requirements

The quality of the water to be produced by the WHWTP must meet all existing and proposed regulatory requirements. The Safe Drinking Water Act (SDWA) of 1974 gave the United States Environmental Protection Agency (USEPA) the authority to set standards for contaminants in drinking water supplies. The USEPA established primary regulations for the control of contaminants that affect public health and secondary regulations for compounds that affect the taste, odor, or aesthetics of drinking water. Drinking water regulations of particular relevance to this surface water supply project include:

- National Primary Drinking Water Regulations (1975)
- Secondary Drinking Water Regulations (1979, 1991)
- Phases I, II and V Regulations (1987, 1991 and 1992, respectively)
- Surface Water Treatment Rule (1989)
- Total Coliform Rule (1989)
- Lead and Copper Rule (1991)
- Consumer Confidence Reports Rule (1998)
- Stage 1 Disinfectants and Disinfection Byproducts Rule (1998) that superseded Total Trihalomethane Rule (1979)
- Interim Enhanced Surface Water Treatment Rule (1999)
- Arsenic Rule (2001)
- Filter Backwash Recycling Rule (2001)
- Long-Term 1 Enhanced Surface Water Treatment Rule (2005)
- Long-Term 2 Enhanced Surface Water Treatment Rule (2006)
- Stage 2 Disinfectants and Disinfection Byproducts Rule (2006)
- Unregulated Contaminants Monitoring Rule – Second Cycle (2006)

Under the provisions or the SDWA, the California Department of Public Health (CDPH) has the primary enforcement responsibility. Title 22 of the California Code of Regulations establishes CDPH's authority and stipulates State drinking water quality and monitoring standards. The CDPH Drinking Water Program is part of the Division of Drinking Water and Environmental Management. The CDPH performs field inspections, issues operating permits, reviews plans and specifications for proposed facilities, enforces compliance with laws and regulations, monitors water quality, and promotes water system security. Furthermore, the CDPH collaborates with the USEPA, State Water Resources Control Board (SWRCB), Regional Water Quality Control Board (RWQCB), and the San Benito County Health Department.

In some cases, the CDPH has yet to establish a state regulation as a companion to a federal regulation, but is responsible for the enforcement of the federal regulation (e.g. Arsenic Rule). On the other hand, the CDPH has established maximum contaminant levels (MCLs) and minimum treatment requirements for surface water that, in some cases, are more stringent than the corresponding federal MCLs, and CDPH has established additional MCLs for several contaminants that are currently unregulated by USEPA. In addition, the CDPH has promulgated state regulations such as the Waterworks Standards that are outside the scope of the federal regulations. The more stringent regulations and MCLs will govern. Appendix F summarizes the CDPH and USEPA MCLs and describes the differences.

A summary of the existing drinking water quality regulations is presented in the balance of this Section and grouped into the following categories:

- Microbial Contaminants
- Disinfectants and Disinfection Byproducts
- Inorganic Chemicals
- Organic Chemicals
- Radionuclides
- Other and Pending Regulations

3.3.1 Microbial Contaminants

3.3.1.1 Total Coliform Rule

Requirements

Under the Total Coliform Rule (TCR), utilities must submit a monitoring plan to CDPH for approval. The plan must provide for representative sampling of the distribution system. The total number of samples and frequency of sampling required is dependent on the population served by the utility. A specific MCL value was not established for total or fecal coliforms under the TCR. Instead, there are three potential scenarios in which an MCL may be violated. These scenarios are:

- If more than one monthly sample proves to be coliform-positive.
- If an original sample is *E. coli* positive, which indicates the presence of fecal material; and if any repeat sample is total, fecal, or *E. coli* positive.

- If an original sample is total coliform-positive and any repeat sample is *E. coli* positive.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

Introducing a properly treated surface water supply into the existing water system is not anticipated to detrimentally affect the City's or SSCWD's monitoring requirements or ability to comply with the TCR.

3.3.1.2 Surface Water Treatment Rule

Requirements

The original Surface Water Treatment Rule (SWTR) was promulgated in June 1989 to further protect customers of public water systems from waterborne infectious diseases. It controls the levels of turbidity, *Giardia lamblia*, viruses, *Legionella*, and heterotrophic plate count (HPC) bacteria in United States drinking waters. The SWTR required all utilities using surface water, or any groundwater supply under the influence of a surface water supply, to provide adequate disinfection, and under most conditions, filtration.

The SWTR includes the following general requirements in order to minimize human exposure to microbial contaminants in drinking water:

- Utilities are required to achieve at least 99.9 percent removal and/or inactivation of *Giardia lamblia* cysts (3-log removal credit) and a minimum 99.99 percent removal and/or inactivation of viruses (4-log removal credit). The required level of removal/inactivation must occur between the point where the raw water ceases to be influenced by surface water runoff to the point at which the first customer is served. Compliance is based on turbidity, disinfectant concentration, and disinfection contact time.
- The disinfectant residual entering the distribution system must not fall below 0.2 milligrams per liter (mg/L) for more than four hours during any 24-hour period.
- A disinfectant residual must be detectable in 95 percent of distribution system samples. An HPC bacteria concentration of less than 500 colonies per milliliter (mL) can serve as a detectable residual if no residual is measured.
- Each utility must perform a watershed sanitary survey at least every five years.
- Additional requirements for filtered supplies.
- These rules are amended by the Enhanced Surface Water Treatment Rule (see below).

3.3.1.3 Interim Enhanced Surface Water Treatment Rule

Requirements

The Interim Enhanced Surface Water Treatment Rule (IESWTR) was promulgated in 1998 and amends the existing SWTR to strengthen microbial protection, including provisions to specifically address *Cryptosporidium*, and to address risk trade-offs with DBPs. The final rule includes treatment requirements for waterborne pathogens, e.g., *Cryptosporidium*. In addition, systems must continue to meet existing requirements for *Giardia lamblia* and viruses. Specifically, the rule includes:

- MCLG of zero for *Cryptosporidium*
- 2-log *Cryptosporidium* removal requirements for filter systems, granted for systems with combined filter effluent (CFE) turbidity $\leq$ 0.3 nephelometric turbidity units (NTU) in 95 percent of samples, never exceeding 1 NTU
- Strengthened combined filter effluent turbidity performance standards
- Individual filter turbidity monitoring provisions
- Disinfection profiling and benchmarking provisions
- Systems using groundwater under the direct influence of surface water now subject to the new rules dealing with *Cryptosporidium*
- Inclusion of *Cryptosporidium* in the watershed control requirements for unfiltered public water systems
- Requirements for covers on new finished water reservoirs
- Sanitary surveys, conducted by States, for all surface water systems regardless of size

The rule, with tightened turbidity performance criteria and individual filter monitoring requirements, is designed to optimize treatment reliability and to enhance physical removal efficiencies to minimize the *Cryptosporidium* levels in finished water. Turbidity requirements for combined filter effluent remain at least every four hours; however, continuous monitoring is required for individual filters.

CDPH Requirements

While the federal rule allows grab sampling in 4-hour intervals, CDPH requires *continuous* monitoring, data recording every fifteen minutes, with on-line turbidimeters, except in cases of equipment failure, for no greater than 48 hours. In addition to the federal 0.3 NTU 95th percentile limit, the CDPH prohibits the average daily CFE turbidity measurement from exceeding 0.2 NTU and requires continuous on-line measurement of individual filter effluent (IFE) turbidity with data recording every fifteen minutes. Grab sampling every four hours is allowed in the case of on-line equipment failure, but is limited to five working days after equipment failure. CDPH requires notification in case of the following exceptions:

- IFE turbidity exceeding 1.0 NTU in two consecutive measurements, 15 minutes apart, at any time during filter operation.
- IFE exceeding 0.3 NTU in two consecutive measurements, 15 minutes apart, after one hour of filter operation following backwash.

- IFE turbidity exceeding 1.0 NTU in two consecutive measurements, 15 minutes apart, at any time during filter operation for three consecutive months.
- IFE turbidity exceeding 2.0 NTU in two consecutive measurements, 15 minutes apart, at any time during filter operation for two consecutive months.

Upon notification of such exceptions, the CDPH will work with water provider to implement corrective measures and preferably avoid an MCL violation.

Disinfection Credits

Cryptosporidium, *Giardia* and virus removal credits are a function of the treatment processes and predicated on compliance with the CFE and IFE turbidity standards. A conventional filtration plant is likely to achieve 2.0-log, 2.5-log and 2.0-log removal credit for *Cryptosporidium*, *Giardia* and viruses, respectively. The required additional disinfection credits would be achieved by chemical inactivation (e.g. chlorination).

The disinfection credit provided by chemical inactivation is a function of disinfectant concentration and effective contact time, typically abbreviated as CT. In a disinfection vessel such as a chlorine contact basin, the effective contact time is determined by a tracer study and defined as the T_{10}, or the time for the tracer concentration at the outlet to reach ten percent of the concentration at the inlet following activation of the tracer dosing system. Water pH and temperature are additional factors in the CT calculation.

Compliance with disinfection performance standards is predicated on:

- Continuous monitoring of disinfectant residual at the entry point to the distribution system; in the case of chlorination, the minimum disinfectant residual at the entry point to the distribution system is 0.2 milligrams per liter (mg/L).
- Measurement of a detectable disinfectant residual in at least 95 percent of distribution system samples collected under the terms of the TCR.
- Daily calculation of minimum CT based on the highest plant flow rate, minimum volume in the contact basin(s), minimum disinfectant residual at the outlet of the contact basin, and maximum water pH and minimum water temperature within the contact basin.

The minimum CT achieved is compared to the CT required each day to confirm compliance with the inactivation requirements. Under the terms of the IESWTR and the Stage 1 Disinfectants/ Disinfectant By-Product Rule, disinfection profiling and benchmarking is required if a change in the disinfection process to reduce DBP formation has potential to compromise microbial protection.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

Compliance with the *Cryptosporidium*, *Giardia* and virus removal and/or inactivation requirements is readily achievable in the proposed WHWTP. The supervisory control and data acquisition (SCADA) system will record CFE and IFE turbidity, calculate CT, and generate monthly reports for submission to CDPH. The WHWTP design will incorporate sufficient control measures to achieve compliance with the SWTR and IESWTR.

3.3.1.4 Filter Backwash Recycling Rule

Requirements

The Filter Backwash Recycling Rule (FBRR) was promulgated by the USEPA in 2001 to control reentry of pathogens and other contaminants into the drinking water treatment process. The key requirements of the rule include:

- Recycle streams must be returned ahead of the portion of primary coagulant addition.
- Direct filtration plants may be required to provide additional information and make modifications deemed necessary.
- Conventional plants that practice direct recycle and have less than 20 filters must perform a one time, one month long self assessment.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The FBRR requirements are addressed in the new WHWTP design.

3.3.1.5 Long Term 1 Enhanced Surface Water Treatment Rule

Requirements

The Long Term 1 Enhanced Surface Water Treatment Rule (LT1ESWTR) became effective in January 2005. The requirements are the same as for the IESWTR, but they also include systems serving populations less than 10,000. The following requirements are included in the rule:

- Requirements based on source water Giardia. (For conventional filtration treatment, typical inactivation/removal of Giardia is 2.5 log. Additional disinfection credits would be achieved through disinfection) :
  - 3-log inactivation/removal of *Giardia* if source water levels < 1 cyst/100 L
  - 4-log inactivation/removal of *Giardia* if source water levels < 9 cysts/100 L
  - 5-log inactivation/removal of *Giardia* if source water levels < 99 cysts/100 L
  - 6-log inactivation/removal *of Giardia* if source water levels > 99 cysts/100 L
- MCLG for Cryptosporidium of zero.
- Filter systems must achieve a 2-log removal of Cryptosporidium.
- Surface water or groundwater systems under the influence of surface water must achieve, through disinfection alone, at least 0.5-log inactivation or Giardia and a 4-log inactivation of viruses.
- Combined filter effluent turbidity requirements for conventional filtration.
- Individual filter monitoring requirements:

- Record continuous monitoring of individual filter performance every 15 minutes.
- Calibrate turbidimeters using manufacturer recommended procedures.
- If continuous monitoring fails, use four hour sample interval for up to five days for compliance.

3.3.1.6 Long Term 2 Enhanced Surface Water Treatment Rule

Requirements

The LT2ESWTR was promulgated by USEPA in 2006 and requires conventional water treatment plants to monitor source water quality for *Cryptosporidium* for 24 months, and then characterize the raw water quality within classifications (also known as Bins) that correspond to additional disinfection requirements (Table 3-2). Water systems can avoid the monitoring requirements of the LT2ESWTR by providing 99.9997 percent removal and/or inactivation of *Cryptosporidium* (i.e. 5.5-log credit) for filtered surface water supplies or 99.9 percent inactivation (i.e. 3-log credit) for unfiltered surface water supplies.

Table 3-2. Bin Requirements

Bin Number	Average *Cryptosporidium* Concentration	Additional Treatment Requirements for Systems with Conventional Treatment that are in Full Compliance with IESWTR1
1	*Cryptosporidium* <0.075/L	No action
2	0.075 ≤ *Cryptosporidium* < 1.0/L	1-log treatment (systems may use any technology or combination of technologies from toolbox as long as total credit is at least 1-log).
3	1.0/L ≤ *Cryptosporidium* < 3.0/L	2.0-log treatment (systems must achieve at least 1-log of the required 2-log treatment using ozone, chlorine dioxide, UV, membranes, bag/cartridge filters, or in-bank filtration.
4	*Cryptosporidium* ≥ 3.0/L	2.5-log treatment (systems must achieve at least 1-log of the required 2.5-log treatment using ozone, chlorine dioxide, UV, membranes, bag/cartridge filters, or in-bank filtration.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The mandatory testing required by the LT2ESWTR was completed for the existing Lessalt WTP, which is fed by the same source water as the new WHWTP. The Hollister-Sunnyslope Water Treatment Agency completed a Drinking Water Source Assessment and Protection Program for the Lessalt WTP in January 2002 and updated the report in March 2009. No positive tests for *Cryptosporidium* were found during testing. The Lessalt WTP has a Bin 1 classification, and thus it is anticipated that the WHWTP will also be held to the requirements of Bin 1 (no additional action).

3.3.2 Disinfectants and Disinfection Byproducts

3.3.2.1 Stage 1 Disinfectants and Disinfection Byproducts Rule

Requirements

The original Total Trihalomethane Rule (TTHM Rule) was promulgated in 1979 and applies to all public water systems serving populations greater than 10,000. The regulation established an MCL of 100 micrograms per liter (µg/L) for total trihalomethanes (TTHMs) in the distribution system. (Total trihalomethanes include the summation of chloroform, bromodichloromethane, dibromo-chloromethane, and bromoform.) Systems must collect a minimum of four distribution system samples per treatment plant on a quarterly basis. Compliance with the MCL is based on the average concentration of the four quarterly monitoring periods.

The Stage 1 D/DBP Rule became effective in January 2002 for water systems serving greater than 10,000 persons and reduced the TTHM MCL to 80 µg/L, down from the 100 µg/L established in 1979. Additional Stage 1 limits for disinfection byproducts are:

- Total Trihalomethanes (TTHMs): 80 µg/L
- Haloacetic Acids (HAAs): 60 µg/L
- Bromate: 10 µg/L
- Chlorite: 1.0 mg/L

The following maximum residual disinfectant levels (MRDLs) have been established to limit the applied dose of chlorine, chloramines, and chlorine dioxide during drinking water treatment. (MRDLs represent the maximum residual concentration permitted at the consumer's tap.)

- Chlorine: 4.0 mg/L (as Cl2)
- Chloramines: 4.0 mg/L (as Cl2)
- Chlorine Dioxide: 0.8 mg/L (as ClO2)

The USEPA has determined that DBP-precursor materials should be regulated in lieu of regulating unidentified DBPs. TOC serves as a surrogate for precursor material, and therefore, requirements for TOC removal have been established. In order to minimize the level of TOC present at the point(s) of disinfection, the D/DBP Rule requires implementation of treatment techniques such as enhanced coagulation at all conventional water treatment plants to reduce elevated levels of raw water TOC. TOC reduction requirements are shown in Table 3-3.

Table 3-3. Total Organic Carbon Reduction Requirements

Source Water: TOC, mg/L	Source Water: Alkalinity, mg/L as $CaCO_3$		
	0-60	60-120	>120
2.0 – 4.0	35%	25%	15%
4.0 – 8.0	45%	35%	25%
> 8.0	50%	40%	30%

Effect on West Hills, City of Hollister, and SSCWD Water Systems

Based on bench testing and pilot testing, it is anticipated that the WHWTP design will incorporate sufficient control measures to achieve compliance with the Stage 1 D/DBPR. Based on average source water TOC and alkalinity levels, Stage 1 D/DBPR requires that the WHWTP remove 25% of the source water TOC. The current treatment process will be designed to remove significantly more that 25% TOC in order to comply with Stage 2 D/ DBP limits in the distribution system.

3.3.2.2 Stage 2 Disinfectant/Disinfection Byproduct Rule

Requirements

The Stage 2 D/DBP Rule was promulgated by the USEPA in 2006 and builds upon the Stage 1 D/DBP Rule to address higher risk public water systems. The MCLs for HAA5 and TTHM remained the same as under the Stage 1 D/DBPR, but this final rule tightens the compliance monitoring requirements. The Stage 2 D/DBP Rule includes the following provisions:

- Sets a Maximum Contaminant Level Goal (MCLG) for chloroform at 0.070 mg/L.
- Requires the re-establishment of sampling locations to better represent extreme conditions (i.e. maximum water age). Water systems using chlorine conduct a yearlong initial distribution system evaluation (IDSE) to identify monitoring sites with peak DBP levels.
- Requires that no later than eight years after promulgation, systems comply with the current 80/60 TTHM/HAAs standards at each new site as 4-quarter running annual average concentrations at each location, also known as locational running annual averages (LRAAs), rather than system-wide averages, using the same MCLs as the Stage 1 D/DBP Rule.
- Temporarily raises the TTHM/HAAs limits to 120/100 to allow time for utilities to make adjustments to come into compliance with the 80/60 TTHM/HAAs standards.

Stage 2 D/DBP compliance monitoring is required to begin for existing systems by:

- April 1, 2012 for systems serving > 100,000
- October 1, 2012 for systems serving 50,000 – 99,999
- October 1, 2013 for systems service < 50,000

The state may grant up to an additional two years prior to the start of compliance monitoring for systems making capital improvements.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The WHWTP will be constructed and started up following the implementation date for compliance monitoring. Achieving compliance with the Stage 2 D/DBPR is a primary treatment objective of this design and the test results from bench and pilot studies indicate that the proposed design incorporates sufficient control measures to achieve compliance with the Stage 2 D/DBPR.

3.3.3 Inorganic Contaminants

3.3.3.1 Lead and Copper Rule

Requirements

The objective of the Lead and Copper Rule (LCR), promulgated in 1991, is to minimize corrosion of lead- and copper-containing plumbing materials in public water systems by requiring utilities to optimize treatment for corrosion control. The LCR establishes "action levels" in lieu of MCLs for regulating levels of both lead and copper in drinking water. The action level for lead is 0.015 mg/L while the action level for copper is 1.3 mg/L. An action level is exceeded when greater than 10 percent of samples collected from the sampling pool contain lead levels above the action levels. Unlike an MCL, a utility is not out of compliance with the LCR when an action level is exceeded. Exceedance of an action level limit requires a utility to take additional steps to reduce lead and copper corrosion in the distribution system. Issues that affect corrosion control such as variations in pH, alkalinity, and hardness need to be taken into account to ensure the finished water is not corrosive.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The City and SSCWD are assumed to be in compliance with the LCR for the existing groundwater wells. Surface water blending from the Lessalt WTP has not affected compliance with the LCR. Water from the WHWTP will increase surface water blending in the system, though because the two treated surface waters entering the system will be similar in quality and treated water pH, it is not anticipated that the WHWTP will impact compliance with the LCR.

3.3.3.2 Other Inorganic Contaminants

Requirements

The original NPDWR of 1975 established MCLs for several metals and minerals that were considered inorganic contaminants (IOCs). The subsequent Phase II and Phase V drinking water regulations of 1991 and 1992, respectively, and the Arsenic Rule of 2001 established additional MCLs and/or revised the original MCLs for IOCs. Slight differences between the CDPH and federal MCLs for IOCs are listed in Appendix F. A unique requirement with a surface water supply is the need to conduct quarterly sampling and analysis for nitrate and nitrite at the point of entry to the water system.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The introduction of treated surface water from the WHWTP into the City and SSCWD distribution systems is anticipated to have little impact on the system IOC concentrations.

3.3.4 Organic Contaminants

Requirements

All of the MCLs for organic contaminants in the original NPDWR were replaced by subsequent regulations, and the current CDPH and federal MCLs for organic contaminants are listed in Appendix F. Phase I drinking water regulations, promulgated in 1987, established MCLs for eight volatile organic chemicals (VOCs); Phase II regulations, promulgated in 1991, established MCLs for ten VOCs and 18 synthetic organic chemicals (SOCs); Phase V regulations, promulgated in 1992, established MCLs for three VOCs and 15 SOCs. Two of the regulated SOCs (i.e. acrylamide and epichlorohydrin) do not have MCLs but compliance is based on treatment techniques. Furthermore, the CDPH has established three MCLs for SOCs and six MCLs for VOCs that are not regulated by USEPA (Appendix F).

Effect on West Hills, City of Hollister, and SSCWD Water Systems

Source water quality data collected from the Lessalt WTP indicates that introducing a properly treated surface water supply from the WHWTP into the existing water system will not detrimentally affect the monitoring requirements or compliance with the SOC and VOC regulations.

3.3.5 Radionuclides

Requirements

Two MCLs for radionuclides (i.e. combined radium 226 and 228 and gross alpha particle activity) in the original NPDWR are still in effect. However, both CDPH and USEPA have since revised the MCLs for the four other regulated radionuclides. The CDPH MCLs for gross beta particle activity, strontium-90, tritium and uranium prevail over the somewhat different MCLs established by USEPA (Appendix F).

Effect on West Hills, City of Hollister, and SSCWD Water Systems

Source water quality data collected from the Lessalt WTP indicates that introducing a properly treated surface water supply from the WHWTP into the existing water system will not detrimentally affect the monitoring requirements or compliance with the radionuclides regulations.

3.3.6 Other Regulations

3.3.6.1 Consumer Confidence Report Rule

Requirements

The Consumer Confidence Report (CCR) Rule, promulgated in 1998, requires all community water systems to issue annual drinking water quality reports to their customers. The City and the SSCWD have historically prepared separate CCRs that each include summaries of the water quality from City of Hollister wells, SSCWD wells, and the Lessalt WTP.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

With the introduction of additional surface water supply into the City water system, the City and SSCWD will need to incorporate into their CCRs the additional water quality data from the WHWTP.

3.3.6.2 Domestic Water Supply Permit Amendment

Requirements

The CDPH requires each public water system to obtain and maintain in good standing a Domestic Water Supply Permit and to amend its Domestic Water Supply Permit prior to implementing changes to the water system operation.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The City and the SSCWD will need to amend their respective Domestic Water Supply Permits to include treated water supply from the WHWTP. The Owner of the WHWTP will need to obtain CDPH approval to operate the new WHWTP and supply treated surface water. The Domestic Water Supply Permit will include a one-page form and supporting documents such as plans, specifications and an operations manual. It is recommended that the Owner meet and consult with CDPH on a regular basis over the course of the project design phases to address any CDPH concerns in advance.

3.3.6.3 Groundwater Rule

The recently promulgated Groundwater Rule (2006) requires groundwater sources to achieve 4-log virus reduction. It is assumed that the existing groundwater supplies to the water system currently meet this requirement.

3.3.6.4 Source Water Assessment and Protection

Requirements

A required component of a Domestic Water Supply Permit is a source water assessment while a source water protection program is voluntary. A source water assessment includes a delineation of the watershed that could introduce contaminants into the water supply, an inventory of possible contaminating activities that could release chemical and/or microbiological contaminants within the delineated watershed, and a determination of water supply vulnerabilities to possible contaminating activities.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

The Owner of the WHWTP will need to submit a source water assessment and updates every five years as part of the Domestic Water Supply Permit for the surface water supply project. The Hollister-Sunnyslope Water Treatment Agency completed an updated Drinking Water Source Assessment and Protection Program for the Lessalt WTP in March 2009. It is anticipated that the WHWTP will be able to use data from this Source Water Assessment to satisfy the requirements of the permit.

3.3.6.5 Unregulated Contaminants Monitoring Rules

Requirements

EPA uses the Unregulated Contaminant Monitoring (UCM) program to collect data for contaminants suspected to be present in drinking water but that do not have health-based standards set under the SDWA. The Unregulated Contaminants Monitoring Rule – First Cycle (UCMR1), promulgated in 1999 and supplemented in 2000 and 2001, required special sampling by large and small water systems with staggered monitoring schedules from 2001 to 2003, although some sampling continued into 2005. Sample analysis was based on the Drinking Water Contaminant Candidate List. The USEPA has completed its analysis of the resulting data, but has yet to determine whether regulating any of the monitored contaminants will reduce public health risk. Therefore, the UCMR1 has yet to result in additional MCLs that might affect the City's water supplies. The monitored contaminants that are considered most likely to receive federal MCLs are methyl tertiary-butyl ether (MTBE) and perchlorate (CDPH recently established a state MCL for perchlorate of 6 μg/L). The Unregulated Contaminant Monitoring Rule – Second Cycle (UCMR2) was promulgated in December 2006 and requires a select group of water systems to monitor 25 chemicals, all water systems serving more than 10,000 people (e.g. City of Hollister and SSCWD) to monitor 10 chemicals, and 320 selected water systems serving from 10,001 to 100,000 people to monitor 15 additional chemicals from the Drinking Water Contaminant Candidate List. Each affected water system was expected to collect samples for a 12-month period from January 2008 to December 2010. The UCMR2 requirements are based on service area population, not the type of water supply. EPA's proposed UCMR3 is anticipated to affect more than 4,800 public and private utilities across the country. The new rules will apply to utilities that serve more than 10,000 people, as well as a sample of 800 smaller systems selected by the EPA. Announced in April, 2010 by the EPA, the new rules could go into effect as early as 2013. The UCMR3 includes monitoring for up to 28 unregulated contaminants.

Effect on West Hills, City of Hollister, and SSCWD Water Systems

Additional sampling and analysis of the SLR and SJR water supply may be needed to confirm the assumption in this Section that introducing a properly treated surface water supply into the City and SSCWD water systems will not detrimentally affect the monitoring requirements or compliance with MCLs that may result from the UCMR1 and UCMR2. Supplemental monitoring will be required for the UCMR3, which included chlorate and could go into effect as early as 2013.

3.3.6.6 CDPH Waterworks Standards (2008 Revision)

The net effects of the revision to the Waterworks Standards include:

- Greater clarity and less ambiguity in the requirements as the result of reframing and updating the existing regulations;
- Requirements for the purpose of ensuring an adequate quantity of drinking water to supply any new developments or expansions of existing water systems prior to their establishment by requiring a comprehensive evaluation of anticipated demand and available supply; and
- Requirements for the purpose of ensuring that materials with which the drinking water may come into contact during transmission, treatment, and distribution do not contaminate the water by requiring that such materials be certified to have met safety standards.

The Waterworks Standards may have some effect on the WHWTP design criteria (e.g. equipment redundancy) but are generally consistent with industry standards.

3.3.6.7 Pending Regulations

Several pending CDPH and federal regulations may affect the design and operation of the proposed WHWTP:

- Chlorate monitoring is proposed to be included in the UCMR3, along with up to 27 other contaminants, which could go into effect as early at 2013.
- The CDPH has included chlorate in its established health-based advisory levels, called "notification levels", to provide information to public water systems and others about certain non-regulated chemicals in drinking water that lack MCLs. The notification level for chlorate is 0.8 mgL. When chemicals are found at concentrations greater than these levels, certain requirement and recommendations apply. A full list of current notification levels for non-regulated chemicals is attached in Appendix F.
- The CDPH is considering state regulations for cross-connection control and groundwater recharge/reuse and an MCL for chromium-6, which are not anticipated to influence the surface water supply project.

Federal revisions to the TCR are anticipated to:

- Address USEPA's obligation to review and revise, as appropriate, each national primary drinking water regulation at least every six years.
- Address public health risks from water quality degradation in distribution systems. The USEPA is working with a Distribution System Advisory Committee to determine the potential need for TCR revisions and distribution system requirements. As with the original TCR, the introduction of additional membrane treated surface water supply into the City water system is not anticipated to detrimentally affect the City's monitoring requirements or ability to comply with the TCR revisions.

A federal Radon Rule was proposed in 1999 and is anticipated to:

- Limit radon levels in drinking water to 4,000 picoCuries per liter (pCi/L) for water systems in communities that implement multimedia mitigation programs to minimize radon levels in indoor air.
- Otherwise, limit radon levels in drinking water to 300 pCi/L. The USEPA has not established a schedule to promulgate the Radon Rule. Since radon is a volatile gas that is rarely found in surface water supplies, the Radon Rule is not anticipated to influence the surface water supply project.

Furthermore, it is always possible that CDPH or USEPA will promulgate additional regulations that will affect the surface water supply project. The initial WHWTP design can incorporate additional regulatory requirements that take effect prior to WHWTP construction. Future regulatory requirements are best accommodated by designing the WHWTP for adaptability and including space allowance for future plant additions.

3.4 Summary

The delivery of treated surface water into the existing water system will require additional water quality monitoring, data reporting, and compliance with drinking water regulations.

The WHWTP design is anticipated to provide the necessary tools to achieve and maintain compliance with the drinking water regulations. It is recommended that the CDPH be consulted on a regular basis over the course of the project design phases to address any CDPH concerns in advance. It is also recommended to conduct additional water quality monitoring in coordination with SSCWD to further characterize the SJR water supply prior to completion of the WHWTP design and construction. Monthly sampling could be completed from a tap in the Hollister conduit near the RWPS turn out.

It is always possible that CDPH or USEPA will promulgate additional regulations that will affect the surface water supply project. The initial WHWTP design can incorporate additional regulatory requirements that take effect prior to WHWTP construction. Potential future regulatory requirements are best accommodated by designing the WHWTP for adaptability and including space allowance for future plant additions.

4 PROCESS EVALUATION AND PILOT TESTING

The planning and development stages of this project incorporated a multi-step process to determine the preferred treatment strategy for the new WHWTP. This section summarizes the methodology and resulting recommendations of the subsequent pilot testing for the selected treatment technology that provided confirmation of treatment capability. Pilot test results are also reviewed herein. The complete evaluation process is detailed in the *Process Alternative Screening Technical Memorandum* (August 2010), Appendix D, the *Process Update for New WTP Technical Memorandum* (November 2010), Appendix D, and the *Summary of Actiflo*® *Carb Pilot Testing Technical Memorandum* (June 2010), Appendix E.

4.1 Source Water Quality and Treated Water Objectives

A significant element of the process evaluation includes the review and characterization the source water quality and the subsequent determination of the treated water objectives. The WHWTP will receive water from the San Luis Reservoir (SLR) and San Justo Reservoir (SJR). Based on historical operation, typical supply will be from the SLR; however, during peak use periods the SJR backfeeds the Hollister Conduit, and thus the source water quality will change.

Both the SLR and SJR are supplied with water from the CVP. SJR water quality is generally similar to SLR because it originates in the SLR, though it is characterized by higher organics and significantly higher seasonal iron and manganese levels. Limited water quality data from the SJR was collected through grab samples and during recent pilot testing. The available water quality for both SLR and SJR is summarized in the Regulatory Review section of this report. Future revised operation of the SJR may increase the frequency of its supply to the Hollister Conduit (and to the WHWTP) during the months of January to March, in addition to the high demand season of late summer and early fall. This report assumes that the future water supply to the WHWTP will consist of approximately 50 - 70 percent SLR and 30 - 50 percent SJR water.

Based on a review of available source water quality data, regulatory standards, and use of free-chlorine for disinfection, the proposed preliminary treated water quality objectives were established, as listed in Table 4-1.

Table 4-1. Proposed Preliminary Treated Water Quality Goals (from Process Alternative Screening TM, August 2010)

Parameter	Units	Primary MCL	Secondary MCL	Treated Water Goal
Turbidity	NTU	TT[(a)]	-	<0.1
Manganese	mg/L	-	0.05	<0.02
Iron	mg/L	-	0.3	<0.1
Disinfection	mg/L-min	-	-	Exceed CT requirement

Parameter	Units	Primary MCL	Secondary MCL	Treated Water Goal
Distribution System Chlorine Residual (as Cl_2)	mg/L	4.0	-	>1.0
Total Coliform	cfu	5% of samples	-	0
Color	pcu	-	15	<10
Odor	ton	-	3	2.4 (80% of secondary MCL)
TDS	mg/L	-	500	
CaCO3 Precipitation Potential	mg/L as CaCO3	-	-	2 to 10
pH	-	-	6.5-8.5	Approx. 8.0
TOC	mg/L	-	-	< 1.5 [(b)]
TTHM at 14 days	mg/L	0.080	-	0.060 (80% of req't)
HAA5 at 14 days	mg/L	0.060	-	0.045 (80% of req't)

Notes:

a) Treatment technique.

b) TOC concentration goal may be adjusted depending on other variables and constituents. During 2011 pilot testing, the treated water TOC goal was reduced from 1.5 mg/L to 1.2 mg/L.

4.2 Process Alternatives Screening

The water quality objectives are summarized above in the Table 4-1. The screening evaluation of treatment process alternatives for the WHWTP provides the basis for the primary objectives for selecting a treatment process included achieving consistent compliance with water quality objectives and economic feasibility for construction and operation.

Understanding that a variety of processes can produce treated water that meets or exceeds regulatory requirements, the regulatory parameter most influencing process selection for the WHWTP is disinfection byproducts (DBPs). DBPs result from the use of free chlorine as the primary and secondary disinfectant. To consistently meet the established distribution system DBP goals, sufficient organic matter must be removed from the source water such that regulated byproduct formation through disinfection is minimized. Bench testing on the SLR water in May 2010 determined that the conventional pretreatment process of enhanced coagulation does not provide the needed levels of organics reduction. (The Jar Test Report from May 2010 is attached in Appendix G). Thus, the *Process Alternative Screening Technical Memorandum* (August 2010) evaluated four treatment technologies with advanced organics removal capabilities (leading to reduced DBP formation), including:

- MIEX
- GAC
- Actiflo® Carb
- Chloramination (alternative disinfection)

Ten cost and non-cost screening criteria were applied for the evaluation of each treatment process alternative as follows:

- Project Cost
- DBP Formation Control and Precursor Removal
- Reliability and Proven Performance
- Iron and Manganese Removal
- Taste and Odor Removal
- Alternate Source Flexibility
- Operational Complexity
- Distribution System Operations
- Liquid and Solids Residuals Handling and Disposal
- Environmental Factors

The alternatives were ranked according to how well they meet each criterion and the screening criteria were weighted to reflect their relative importance to one another. As part of the screening evaluation, capital, operation and maintenance, and present value costs were developed for each of the alternatives.

The results of the screening evaluation concluded that the highest ranked alternatives were 1) Conventional Treatment with GAC contactors, (ranked highest), and 2) Actiflo® Carb with gravity filtration (ranked second highest). A subsequent sensitivity analysis performed on the two recommended alternatives indicated that the GAC alternative would fall to second ranked behind Actiflo® Carb if the required GAC exchange frequency increased to every five months or less.

Based on the outcome of the screening evaluation, representatives from HDR and the MOU Parties participated in a tour of the Palmdale WTP, in Palmdale, California, where GAC contactors are used.

Figure 4-1. Palmdale WTP GAC Contactors

The plant data and operator information collected during the site visit confirmed that for GAC to be used effectively at the WHWTP, the GAC exchange frequency could be as often as every three and a half months. Therefore, the Actiflo® Carb process became the highest ranked alternative and was ultimately recommended for confirmatory pilot testing.

4.3 Actiflo® Carb Pilot Testing

This section summarizes the objectives and findings of the pilot testing that was conducted for the Actiflo® Carb pretreatment system. The overall objective of the pilot test was to define and demonstrate the optimized treatment of site-specific source waters using the tested system. The results of the pilot test serve as a guideline for the full-scale application of Actiflo® Carb at the WHWTP. The operating parameters, testing, sampling, and analysis of the pilot system were established to be indicative of full-scale operation.

Pilot testing of the Actiflo® Carb system was conducted at the Lessalt WTP during February and March of 2011.

The three phases for the pilot test included:

- Phase 1: San Luis water supply
- Phase 2: San Justo water supply
- Phase 3: Combination supply: various blends of San Luis and San Justo Water

Figure 4-2. Actiflo® Carb Pilot Unit

Each phase of testing included the optimization of chemical feed rates, simulated distribution system (SDS) testing, confirmation that TOC removal goals, and evaluation of pre-oxidants upstream of the Actiflo® Carb system.

The testing concluded that the Actiflo® Carb system (upstream of gravity filters) *demonstrates* the treatment capability and flexibility to meet DBP goals for both SLR and SJR water sources. The pilot test established optimal operating parameters and associated chemical dose ranges for treatment of each source water. These design criteria were incorporated into the preliminary design proposal for the system, which is attached in Appendix A.

As a follow up to the pilot testing, additional bench scale testing is recommended during the fall 2011 season when more challenging source water quality (i.e., elevated iron, managanese, and organics) is typically received at the Lessalt WTP and similarly anticipated at the WHWTP. The objectives of the bench testing include:

- Confirmation of iron and manganese removal through the use of pre-oxidants, and
- Confirmation of treatment and optimized chemical dose ranges for the more challenging seasonal water quality.

Additional details regarding the specific testing objectives, test plan and protocol, results, and conclusions are included in the *Summary of Actiflo® Carb Pilot Testing Technical Memorandum* (June 2010), which is attached as Appendix E of this PDR.

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5 RAW WATER PUMP STATION AND PIPELINE

5.1 Introduction

This section presents design criteria for the Raw Water Pump Station (RWPS) and pipeline that will deliver water to the WHWTP. The pump station design will be based on supplying surface water to the WHWTP at the initial capacity of 6.0 MGD with provisions to accommodate expansion to the final capacity of 9.0 MGD.

The WHWTP will utilize water in the Hollister Conduit for the purpose of supplying drinking water to the HUA. The Hollister Conduit is supplied by water from both the SLR and SJR. Future supplies may include the North County water bank. The source of the feed water to the RWPS will be from a new tap in the Hollister Conduit.

5.2 RWPS Site

Two sites were evaluated for the pump station location. The location options for the RWPS are shown in Figure 5-1.

Figure 5-1. Raw Water Pump Station Site Options

The first site location is at the intersection of Richardson Road (the WHWTP access road) and Union Road. The primary advantage of this site is its proximity to the WHWTP, which results in a shorter raw water pipeline. Approximately one acre of land would need to be acquired.

The second potential site for the RWPS is at the location of the Hollister Conduit sectionalizing valve facility. The valve is approximately 0.8 miles southeast of the WHWTP location along Union Road. The site is currently owned by the SBCWD and use of this space would eliminate the need for additional property acquisition.

Table 5-1 below reviews the advantages and disadvantages of each site. As shown in the table, the additional length of raw water pipe that would be required for use of the sectionalizing valve site is far greater than the acquisition of a new, closer site. Therefore, the access road site is recommended for the RWPS. The site layout would provide sufficient setback from Union Road for safety and aesthetic considerations. Fencing along the perimeter would provide site security with primary access from Union Road. It is recommended that the MOU Parties actively pursue acquiring this additional property.

Table 5-1. RWPS Site Selection

Site Location	Advantages	Disadvantages
Sectionalizing Valve Site	SBCWD Currently Owns the Property Already fenced for security	Additional 20" Raw Water Pipe Cost- $703k Additional pipe length : 3,700 ft
Access Road Site	Total 20" Raw Water Pipe Cost- $500K	Site Would Need to be Acquired- Cost Approximately $100K (1 Acre)

The RWPS site will also contain a chemical storage and feed facility for liquid sodium permanganate ($NaMnO_4$). Sodium permanganate will be added periodically in low doses to the raw water to oxidize dissolved iron and manganese so that it can be readily removed at the WTP. The chemical building will be approximately 80 sf, constructed of split face concrete masonry unit (CMU) block, and will store two totes (300-400 gallon tanks) of sodium permanganate and the associated feed pumps. Additional information on the sodium permanganate dosing and storage is provided in Section 8 - Chemical Storage and Feed.

5.3 RWPS Capacity and Design

The RWPS is designed to deliver a range of flows from 1.5 to 6.0 MGD of untreated water to the WHWTP at the initial phase and will be expandable to 9.0 MGD at the final phase. The initial phase is expected to be in operation in less than three years. The RWPS will be located outside and the associated electrical panels will be located adjacent to the pumps in a weatherproof enclosure.

The pumps will be end suction centrifugal pumps, mounted on a concrete pad at grade. The RWPS will consist of four (three initially, four ultimately) equally sized pumps equipped with variable frequency drives (VFDs). This will allow efficient operation as required to maintain flow and target water surface levels in the pretreatment (Actiflo® Carb) basins. The VFDs will also accommodate low-flow conditions. The RWPS is sized to accommodate the worst-case water level elevation in the SJR. The minimum SJR water level is 444 feet. The pump station discharge will be measured by a magnetic-type flow meter located adjacent to the pumps. Air release valves will permit air to escape from the pumps suction and pipeline. Design criteria for the RWPS are listed below. A preliminary plan and section drawing of the RWPS is included in Appendix B as Figure B-5.

Potable water service will be supplied to the RWPS by a small diameter tie-in to the distribution pipeline from the WHWTP.

A new 480/277-volt utility service is required to serve the RWPS. Electrical service will be sized to meet the demands at the build out capacity of 9.0 MGD. A transfer switch will be installed and conduits stubbed to the outside for a future generator.

Security measures will be provided at the new site to protect the facility from vandalism or other threats to the water supply. Secure locks and intrusion alarms will be provided for the doors and electrical panels. Lighting and video cameras will be provided at the building that will have the ability to record and store up to 24-hours of data.

5.3.1 Initial Phase

- Total pumping capacity: 9 MGD (3 pumps @ 3 MGD each)
- Firm pumping capacity: 6 MGD (2 pumps @ 3 MGD each, 1 standby)
- Total design head (TDH): 60 feet (ft) static + 25 ft pipe head loss = 85 ft TDH
- End suction pump sizes:
  - Pump 1 3 MGD (75 hp, VFD)
  - Pump 2 3 MGD (75 hp, VFD)
  - Pump 3 3 MGD (75 hp, VFD)
  - Pump 4 Empty Pump Pad

5.3.2 Final Phase

- Total pumping capacity: 12 MGD (4 pumps @ 3 MGD each)
- Firm pumping capacity: 9 MGD (3 pumps @ 3 MGD each, 1 standby)
- Total design head: 60 ft static + 50 ft pipe friction head loss = 110 ft TDH
- End suction pump sizes:
  - Pump 1 3 MGD (75 hp, VFD)
  - Pump 2 3 MGD (75 hp, VFD)

- Pump 3 3 MGD (75 hp, VFD)
- Pump 4 3 MGD (75 hp, VFD)

5.4 Raw Water Pipeline

A 20-inch diameter pressurized raw water pipeline will extend northeast from the RWPS site, along the 25-foot wide Richardson Road easement to the treatment plant inlet structure. The pipeline will be either ductile iron or steel with a bury depth of approximately four feet.

The pipe outlet elevation at the RWPS discharge is approximately 350 feet, and the termination elevation of the raw water pipeline at the inlet structure to the WHWTP is 504 feet. The total dynamic head for the pumps is 85 feet (initial) and 110 feet (build out). Plan and profiles of the pipeline alignment will be developed during final design. A preliminary plan of the raw and treated water pipeline alignment is attached in Appendix B as Figure B-6.

The velocity in the 20-inch diameter raw water pipeline will be 4.2 feet per second (fps) under initial conditions with a design flow of 6.0 MGD. Under the build out condition of 9.0 MGD, the velocity in the 20-inch diameter raw water pipeline will be 6.3 fps. Surge conditions in the pipeline will exist during the periodic shutdown of the pump station. An adequate surge arrestor will be installed on the discharge header to accommodate such events, and the exact surge control methodology will be determined during final design.

Construction of the pipeline will include site clearing and construction of a 20 foot wide temporary gravel access road along the pipeline alignment for installation of the pipe.

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6 PRETREATMENT

Pretreatment at the WHWTP includes all processes that occur upstream of filtration. To achieve the overall regulatory requirements, as well as the plant specific treatment goals established during the water quality and process screening, the following specific pretreatment strategies for the WHWTP are planned:

- Addition of a pre-oxidant for the precipitation (and later removal through filtration) of seasonal dissolved iron and manganese in the source water.
- Addition of powder activated carbon (PAC) for the adsorption and removal of organic precursor material (measured by the surrogate parameter TOC) to control DBP formation.
- Coagulation, flocculation, and sedimentation for supplemental removal of organic precursor material (measured by the surrogate parameter TOC) to control DBP formation and remove turbidity prior to filtration.

This Section discusses the above pretreatment approach, including the application of the selected advanced organics removal process (Actiflo® Carb) at the WHWTP.

6.1 Preoxidant Alternatives for Iron and Manganese Removal

Periodic elevated iron and manganese concentrations from the SJR have been sampled at levels as high as 1.7 milligram per liter (mg/L) of iron and 0.68 mg/L of manganese. The SJR has been observed to produce high iron and manganese events when the dissolved oxygen (DO) level in the SJR reservoir drops. In these low-oxygen conditions, the iron and manganese are typically in a dissolved, soluble form. As noted in the *Lessalt Oxidant Evaluation TM* (March 2011), historical dissolved iron and manganese conditions occur several times per year at the Lessalt WTP for approximately one to two weeks. The future reservoir management strategy may include operating SJR at low DO levels to facilitate control of the zebra mussels in the Hollister Conduit; therefore, future concentrations of iron and manganese may increase. During periods when the DO is low in SJR, both iron and manganese can exceed their respective State of California secondary MCLs of 0.3 mg/L and 0.05 mg/L at the Lessalt WTP.

To meet the secondary MCLs and the established treated water goals for iron and manganese at the WHWTP, treatment is required for a reduction of the iron and manganese to desirable levels less than approximately 0.1 mg/L and 0.02 mg/L, respectively. The addition of a pre-oxidant to the raw water will assist with iron and manganese removal. An evaluation of potential pre-oxidation chemicals follows.

6.1.1 Chlorine

Chlorination may be performed using chlorine gas or other chlorinated solutions that may be in liquid such as sodium hypochlorite. Although the primary use of chlorination is disinfection, it also serves as an oxidizing agent for taste and odor control, prevention of algal growth, maintaining clear filter media, removal of iron of the distribution systems, and improving coagulation. When added to the water, free chlorine reacts with NOM and bromide to form DBPs, primarily trihalomethanes (THMs), some HAAs, and others.

Chlorine is often applied to raw water in conjunction with coagulants, at the end of the clarifiers, upstream of filters, post-filtration (disinfection), or in the distribution system. Because preoxidant dosing and contact occurs upstream of the Actiflo® Carb system, it is anticipated that any remaining chlorine residual would be adsorbed by the PAC within the system.

6.1.2 Potassium Permanganate

Potassium permanganate is typically supplied in dry form in buckets, drums, and bins. A diluted solution of potassium permanganate is generated on-site using a day tank, plant water, feed equipment, and a mixer. Potassium permanganate serves as an oxidizing agent to control odor and taste, remove color, control biological growth, and remove iron and manganese. Potassium permanganate typically requires 10-20 minutes of contact time for oxidation. Pretreatment with permanganate in combination with post-treatment chlorination will typically result in lower DBP concentrations than would otherwise occur from pre-chlorination.

Potassium permanganate is typically applied to raw water in conjunction with or upstream of coagulants addition or filtration. Special care should be used to not over feed the permanganate, as it can add manganese and color to the water. Because preoxidant dosing and contact will occur upstream of the Actiflo® Carb system, any remaining permanganate residual resulting from a potential overdose will be adsorbed by the PAC within the system.

6.1.3 Sodium Permanganate

Sodium permanganate is supplied as either a 20 or 40 percent liquid solution and is stored in a bulk storage tank. The solution would be fed through metering pumps to the raw water supply. Similar to potassium permanganate, sodium permanganate serves as an oxidizing agent to control odor and taste, remove color, control biological growth, and removes iron and manganese and requires 10-20 minutes of contact time. Pretreatment with permanganate in combination with post-treatment chlorination will typically result in lower DBP concentrations than would otherwise occur from pre-chlorination.

Sodium permanganate is typically applied to raw water in conjunction with or upstream of coagulants addition or filtration. Special care should be used to not over feed the permanganate, as it can add manganese and color to the water. Because preoxidant dosing and contact will occur upstream of the Actiflo® Carb system, any remaining permanganate residual resulting from a potential overdose will be adsorbed by the PAC within the system.

6.1.4 Chlorine Dioxide

Chlorine dioxide must be generated on-site. In most potable water applications, chlorine dioxide is generated as needed and directly educed or injected into a diluting stream. Generators are available that utilize sodium chlorite and a variety of feedstock, such as chlorine gas, sodium hypochlorite, and sulfuric or hydrochloric acid. Chlorine dioxide is utilized as a primary or secondary disinfectant, for taste and odor control, THM/HAA reduction, iron and manganese control, color removal, sulfide and phenol destruction, and Zebra mussel control.

When added to water, chlorine dioxide reacts with many organic and inorganic compounds. The reactions produce chlorite and chlorate as end products, compounds that are suspected of causing hemolytic anemia and other health effects. However, chlorine dioxide does not produce THMs. The use of chlorine dioxide aids in reducing the formation of THMs and HAAs by oxidizing precursors.

Chlorine dioxide used for oxidation is typically fed into the raw water, sedimentation basins, or following sedimentation. The dosage is typically limited to 1.4 milligrams per liter (mg/L) in order to limit the total combined concentration of ClO_2, ClO_{2-}, and ClO_{3-} to a maximum of 1.0 mg/L. Under the U.S. Environmental Protection Agency (EPA) DBP regulations, the maximum residual disinfection level (MRDL) for chlorine dioxide is 0.8 mg/L and the MCL for chlorite is 1.0 mg/L. Since preoxidant dosing and contact occurs upstream of the Actiflo® Carb system, it is anticipated that any remaining chlorine dioxide residual would be adsorbed by the PAC within the system.

6.1.5 Pre-oxidant Comparison and Selection

As a pre-oxidant, chlorine is generally less effective than potassium and sodium permanganate, which is less effective than chlorine dioxide. Conversely, when considering capital, operations, and maintenance (O&M) costs, chlorine is generally more cost effective than sodium permanganate and potassium permanganate, which is more cost effective than chlorine dioxide. Table 6-1 provides a relative general comparison of the pre-oxidant options.

Table 6-1. Pre-oxidant Alternative Comparison

Disinfectant	Reduce DBP Precursors	Taste and Odor Control	Iron Control	Manganese Control	Color Control	Cost
Chlorine	P	G	E	F	G	E
Potassium Permanganate	G	G	G	E	F	G
Sodium Permanganate	G	G	G	E	F	G
Chlorine Dioxide	E	G	E	E	G	P

Relative Comparison; P=Poor, F=Fair, G=Good, E=Excellent

Based on the analysis presented above, either sodium permanganate or potassium permanganate is preferred. The preoxidant will be injected at the raw water pump station to allow sufficient contact time. According to Carus Corporation, one of the leading permanganate suppliers for water treatment facilities, newer water treatment plants are trending towards the use of liquid sodium permanganate over potassium permanganate for pre-oxidation. Historically, the use sodium permanganate in California presented a higher operational cost due to the proximity of the closest plant in Illinois and the resulting cost of delivery. However, Carus Corporation will be opening a new facility in California in the next few years which will significantly reduce delivery costs. Additionally, the capital cost of a potassium permanganate system is significantly greater due to the additional equipment required including a mix tank, feed pump, dust collection system, hoist for bulk bags, etc. The liquid system only requires a bulk storage tank and metering pumps.

The existence of multiple suppliers of sodium permanganate ensures that the chemical is not supplied by a single company and thus provides protection in future operations to ensure costs are competitive. Other reputable sodium permanganate suppliers include Hepure Technologies (Wilmington, DE and Berkeley, CA) and Altiva (Houston, TX and San Gabriel, LA).

Storage sizing and permanganate feed information is provided in Section 8 - Chemical Feed and Storage. Details regarding requirements for the permanganate storage building are included in Section 5 - Raw Water Pump Station.

6.1.6 Iron and Manganese Removal within the Actiflo® Carb and Filter System

Following the oxidation process, particulate iron and manganese will be removed together with the sludge discharge from the Actiflo® Carb system. Carryover iron and manganese from the Actiflo® Carb system will be removed through the filters. During pilot testing, it was learned that iron based coagulants such as ferric chloride as used in this test, frequently contain trace amounts of manganese. Since the PAC in the Actiflo® Carb system will remove any remaining permanganate residual dosed at the RWPS, removal of the trace amounts of (coagulant-added) manganese would occur through filtration. Though the test runs during piloting were inconclusive due to low raw water levels of Fe/Mn, the full-scale WTP is expected to achieve the established Fe/Mn removal goals.

Iron removal through use of a pre-oxidant (either permanganate or ClO2) is an established and well documented process in full scale WTPs throughout California. Iron removal in the full scale system is expected to achieve the project's target goals as established in the *Water Quality and Process Alternative Screening TM* (August 2010). Obtaining confirmation of iron removal is anticipated during future recommended bench tests on raw water collected during elevated iron water quality episodes at the Lessalt WTP.

During full scale system operation, the gravity filters downstream of the Actiflo® Carb unit will be operated with a minimal chlorine residual through the filters. Under these conditions, the filter media develops a manganese oxide coating which enhances the removal of dissolved manganese. It is anticipated that the trace amounts of manganese added from the coagulant will be removed though the filters.

6.2 Actiflo® Carb System

The Actiflo® Carb system provides pretreatment upstream of the gravity filters. Control of the influent water flow from the RWPS is based on maintaining a target water level within the Actiflo® Carb basin. The pretreatment basin will be concrete construction and will be adjoined to the downstream gravity filters.

6.2.1 Process Description

The Actiflo® Carb system combines the traditional Actiflo® ballasted microsand clarification process with the recirculation of powdered activated carbon (PAC) to enhance the removal (adsorption) of organic matter (measured as TOC) and taste and odor. The microsand serves several important roles in the ACTIFLO® process such as forming a seed which promotes the formation of large stable, high-density floc, and dampening the effects of changes in the raw water quality due to its high concentration within the process. The microsand is effectively removed from the chemical sludge and reused in the process due to its chemically inert qualities. The ballasted floc has considerably higher settling velocities than conventional floc and allows significantly higher clarifier overflow rates. Figure 6-1 depicts a schematic of the complete Actiflo® Carb system.

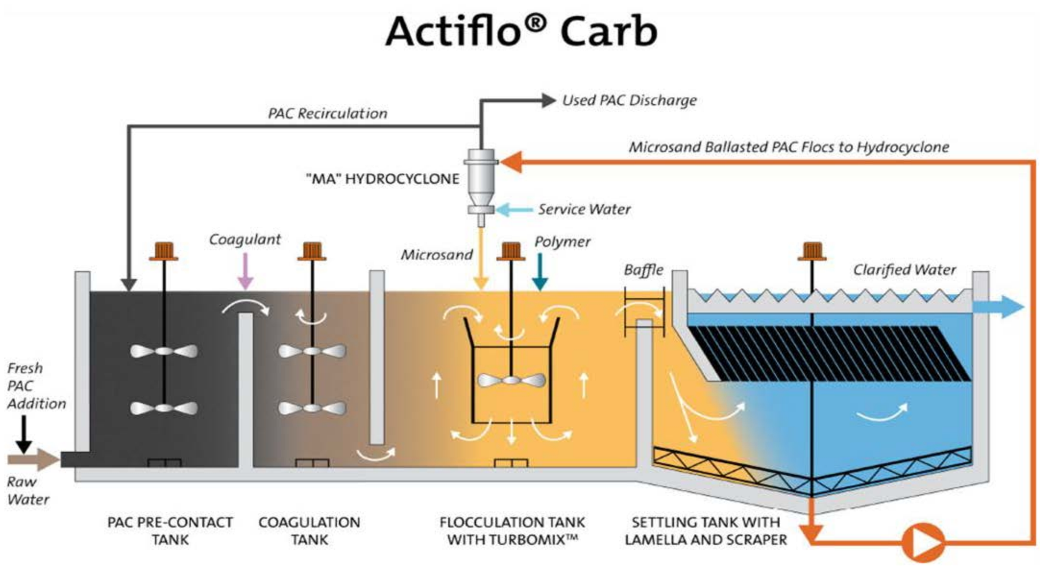

Figure 6-1. Actiflo® Carb System Schematic

The recirculated PAC provides enhanced adsorption of natural organic molecules while allowing for reduced operating costs due to lower PAC usage requirements. The resulting PAC and sand ballasted floc display unique settling characteristics, which allow for clarifier designs with high overflow rates and short retention times. Thus, the Actiflo® Carb replaces conventional flocculation / sedimentation basins with a smaller footprint. Since feed turbidity is consistently < 10 ntu, Actiflo® Carb does not require upstream clarification to optimize the

PAC adsorption capacity use by soluble organic matter rather than solids. Filtration follows the Actiflo® Carb process.

The Actiflo® and Actiflo® Carb processes are currently in operation worldwide in small communities as well as large metropolitan areas. A list of current system installations is attached in Appendix H.

Several advantages that Actiflo® Carb process offers compared to conventional treatment alternatives include:

- Large sand particle surface area serve as a "seed" for floc formation which when combined with polymer produces a large, stable floc with fast settling characteristics.
- Enhanced coagulation allows for variable process chemistry with efficient chemical usage.
- Recirculated PAC provides enhanced adsorption of natural organic molecules while allowing for reduced operating costs due to lower PAC usage requirements.
- Flexible and stable process for handling fluctuating water quality while continuously delivering high quality effluent.
- Rapid start-up and shut-down with a short hydraulic retention time.

6.2.2 System Reliability

Two 3 MGD units will be constructed for the initial plant capacity of 6 MGD. A third 3 MGD unit will be added upon plant expansion to 9 MGD. There are few moving parts to the Actiflo® Carb system, which minimizes the need for redundant system units. A recommended replacement parts list (including motors, drives and mixer blades), the lead time for each part, the time it would take to maintenance the parts once onsite, and the price for the parts is attached in Appendix H. Incorporating these selected parts as spare parts would minimize plant down time for potential repairs or replacements. Supplemental information provided by Kruger regarding the system's inherent prevention of hydrocyclone clogging, and mixer or scraper mechanism breakdown is also attached in Appendix H.

6.2.3 Single Supplier

Because the preferred pretreatment process, Actiflo® Carb, is proprietary and is manufactured and marketed by Kruger, the equipment bid must be negotiated with Kruger, rather than competitively bid. Thus, it is essential to maximize negotiation opportunities for pricing throughout the planning and design phases. Kruger provided system proposals and budget pricing initially in April 2010 and later (based on the pilot test results) provided revised proposals in April 2011, May 2011, and June 2011. Negotiations on scope and pricing will continue during final design and can be finalized (to prevent a potential increase) once the agreements between the MOU Parties are final and the project's future is imminent.

6.2.4 Supplemental Testing Recommendations

As a follow up to the February and March 2011 pilot testing for Actiflo® Carb, and as further confirmation of the established design parameters, supplemental bench scale testing is recommended during the fall 2011 season when the more challenging SJR source water quality (elevated iron, manganese, and organics) is fed to the Lessalt WTP. Bench testing could be conducted by the Lessalt WTP operations staff and/or SBCWD staff, with assistance from Kruger field staff. The bench tests will include confirmation of iron and manganese removal through use of a pre-oxidant, establishing oxidant demand, and simulation of the Actiflo® Carb process to confirm treatment parameters for the more challenging water quality.

6.2.5 Design Criteria

Table 6-2 summarizes the design criteria for the proposed Actiflo® Carb system.

Table 6-2. Actiflo® Carb Design Criteria

Design Parameter	Unit	Initial Phase – 6 MGD	Future Phase – 9 MGD
Design Flow per Unit	MGD	3	3
Number of Units	number	2	3
PAC Contact Tank HRT	min	8.0	8.0
Coagulation Tank HRT	min	2.0	2.0
Maturation Tank HRT	min	4.5	4.5
Rise Rate	gpm/ft^2	13.5	13.5
Sand Recirc. Pumps per Unit	number	1 duty + 1 standby	1 duty + 1 standby
San Recirculation Pump Capacity	gpm	190	190
Number of Hydrocyclones per pump	Number	1	1
Estimated Sludge Concentration	% solids	0.5	0.5
Est. Sludge Discharge per Unit at Design Flow	gpm	1-2	1-2
PAC Contact Tank			
Length	ft	13’ 2”	13’ 2”
Width	ft	15’ 5”	15’ 5”
Side Water Depth	ft	12’ 0”	12’ 0”
Coagulation			
Length	ft	7’ 1”	7’ 1”
Width	ft	7’ 1”	7’ 1”
Side Water Depth	ft	12’ 0”	12’ 0”
Maturation Tank			
Length	ft	9’ 3”	9’ 3”
Width	ft	12’ 4”	12’ 4”
Side Water Depth	ft	12’ 0”	12’ 0”
Settling Tanks			
Length	ft	15’ 5”	15’ 5”
Width	ft	15’ 5”	15’ 5”
Side Water Depth	ft	12’ 0”	12’ 0”

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7 FILTRATION

Downstream of the pretreatment (preoxidant, PAC contact, coagulation, flocculation, and sedimentation), the filtration system at WHWTP provides supplemental removal of turbidity, coagulated organic material, and oxidized particulate iron and manganese. The chemical-physical process of oxidation of metal oxides for removal through filtration is a proven and well-established treatment technology. The filtration system will use anthracite and filter sand media. A chemical dose point for coagulant will be located downstream of pretreatment and upstream of the filters. A minimal dose of up to 1 mg/L will chemically "tie-up" any remaining polymer that may carry over from the Actiflo® Carb process and thus improve filter performance.

7.1 Filtration Process Options

Three filtration options were considered for the WHWTP facility.

- Conventional gravity filtration (packaged metal structure)
- Conventional gravity filtration (concrete structure)
- Pressure filtration

The filter design is based on historical water quality information, as summarized in the Regulatory Review Section of this report and the *Process Alternatives Screening Evaluation TM* (August 2010).

7.1.1 Conventional Gravity Filtration - Package Steel Systems

Package steel filtration systems utilize a single unit package that requires only an inlet/outlet connection for integration into the treatment plant. Similar to the concrete gravity and pressure filters, the package system is capable of removing turbidity, suspended solids, color, iron, manganese, odor, taste and parasites such as *Giardia lamblia* and *Cryptosporidium*. Table 7-1 below reviews the advantages/disadvantages and cost of the package steel option. Manufacturer information is attached in Appendix I. The package filters are pre-fabricated for simplified installation on a concrete slab. Package filters can provide a cost effective and easily expandable treatment option for smaller WTPs (generally less than 3 MGD, but up to 10 MGD in size). Package filters tend to have a limited life span of 20 to 30 years. In visibly sensitive areas, the aesthetics of the package system may be questionable. Because the Actiflo® Carb system will be concrete construction, the advantages of a package filter system for the WHWTP are minimized.

7.1.2 Conventional Gravity Filtration - Concrete Systems

Concrete filters are field constructed, after which the interior and exterior equipment components are added to the system. Concrete has a life span of up to 50 years, requires less maintenance for painting than steel, and is more economical than steel, based on recent completed projects and current budget pricing. Table 7-1 below summarizes the advantages/disadvantages and cost of the concrete filter option.

7.1.3 Pressure Filtration

Pressure filtration systems utilize pressure vessels to filter the water. They are typically long horizontal cylinders that filter from the top down. The system is capable of removing turbidity, suspended solids, color, iron, manganese, odor, taste and parasites such as *Giardia lamblia* and *Cryptosporidium*. Table 7-1 below reviews the advantages/disadvantages and cost of the pressure filter option. Manufacturer information is attached in Appendix I.

Table 7-1. Filtration Comparison

Configuration	Advantages	Disadvantages	Backwash Generated (6 MGD)	Conceptual Cost		
				6 MGD	9 MGD	Source
Conventional (Packaged)	-All inclusive design -System can operate at up to 6 gpm/sf	-Steel is expensive, relative to concrete -Steel has a shorter life than concrete	371,500 gal/day	$1,340,000	$1,890,000	Budget Quote[(a)]
Conventional (Concrete)	-Concrete has longer life than steel -Requires less maintenance for painting -Takes advantage of concrete costs -Seamless integration with a concrete Actiflo Carb system -System can operate at up to 6 gpm/sf	-A single manufacturer is not responsible for the performance of the entire package	320,000 gal/day	$1,069,000	$1,600,000	Cost Estimate
Pressure	-Clean/low profile -Compartmental design -Can operate at relatively high headloss	-Loading rate is limited to 3.2 gpm/sf -Resulting cost is high	371,500 gal/day	$1,633,000	$2,277,000	Budget Quote[1]

Notes:
a) Includes equipment, tax, installation, concrete slab.

7.1.4 Filtration System Recommendation

All three filtration alternatives provide excellent treatment quality options for the WHWTP. The primary difference between gravity and pressure systems is the required footprint required. The State SWTR limits pressure filters to 3 gpm/sf and gravity filters to 6 gpm/sf. Although approval for higher rates may be sought from CDPH on a case by case basis, it must be demonstrated that the performance at the higher rate is equal or better than at the approved rate. The increased surface area requirement for pressure systems results in increased cost. The design criteria for both conventional package and concrete gravity filter systems is summarized in Table 7-2 below. As noted in Table 7-1, the estimated cost for the conventional concrete gravity filter is the lowest of the three alternatives. Based on relative cost and the analysis summarized in Table 7-1, concrete conventional filters are recommended for use at the

WHWTP. A preliminary plan view drawing of the filters is attached in Appendix B as Figure B-9. A sample photo of a conventional filter is also included below in Figure 7-1.

The layout arrangement for the filter cells includes adjacent cells with common wall construction. The filter gallery will extend along one side of the filter cells. The filters will be located outdoors, though a small filter cell building above the filter gallery will house the local control panels.

Figure 7-1. Conventional Filter

7.2 Backwash Strategy

The backwash requirements vary based on loading, water quality, and treatment goals. Dual media filters are cleaned effectively by backwash supplemented by air scour. Typical backwash rates for gravity dual media filters are 18-20 gpm/sf. Air scour will be included in the backwash process to enhance solids removal.

Each filter will be individually backwashed using treated supplied water from the treated water storage tank. Conventional concrete filters typically backwash once per day. Backwash water production is summarized in Table 7-2.

A concurrent air scouring wash is assumed for the dual media filter. The backwash process begins with air scour only for five minutes, then a slow backwash rate with simultaneous air scour until the water level is one foot below the troughs, followed by a high backwash rate to purge trapped air and re-stratify the media.

7.3 Backwash Pumps

Two backwash supply pumps (1 duty and 1 standby will be provided. The backwash pumps will rotate lead when a backwash cycle is initiated. Each pump will provide a design flow rate equal to the backwash rate required by the filter. It is anticipated that only one filter will be in backwash at a time. Due to the hydraulic profile and site grade, the backwash pump station will be located adjacent to the treated water storage tank. Pump design criteria is summarized on Table 7-2.

Table 7-2. Conventional Design Criteria

Filtration System	Units	Steel Package		Concrete	
		6 MGD	9 MGD	6 MGD	9 MGD
Number of Units	Number	5 (4 active + 1 standby)	7 (6 active + 1 standby)	3	4
Nominal Capacity per Unit	MGD	1.5	1.5	3	3
Length (Each)	ft	8	8	30	30
Width (Each)	ft	22	22	12	12
Height	ft	8	8	16	16
Area per Filter	sf	176	176	360	360
Total Area	sf	704	1056	1080	1440
Filtration Rate	gpm/sf	6	6	3.86	4.34
Max Filtration Rate (w/ 1 unit in bw)	gpm/sf	6	6	5.79	5.79
Air –Scour Rate	cf/min/sf	-	-	4	4
Backwash Rate During Air Scour	gpm/sf	-	-	6 to 8	6 to 8
Air-Scour Duration	min	-	-	5	5
Backwash Rate (maximum) [a]	gpm/sf	30	30	18-20	18-20
Backwash Duration	min	9	9	18	18
Minimum Duration Between Backwashes	hr	12	12	24	24
Total Daily BW Water Usage / Waste Volume	gal	371,500	464,000	324,000	432,000
Backwash pumps	Number	2 (1 duty + 1 standby)	2 (1 duty + 1 standby)	2 (1 duty + 1 standby)	2 (1 duty + 1 standby)
Capacity	gpm @ ft TDH	5,280 @ 30'	5,280 @ 30'	7,200 @ 30'	7,200 @ 30'
Motor	HP	75	75	90	90
Filter Media					
Type	type	Varies per manufacturer		Anthracite/Sand	Anthracite/Sand
Depth	in			20/10	20/10
Size	mm			1.0/0.5	1.0/0.5
Uniformity Coefficient				1.45/1.3	1.45/1.3
FTW					
Duration	Min	20	20	20	20
Total daily volume FTW	Gal	84,500	126,700	83,400	125,000

Notes:
a) Average backwash rate over total duration is 12 gpm/sf.

7.4 Washwater Handling

Washwater handling is discussed in Section 10 - Solids/Backwash Handling.

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8 CHEMICAL FEED AND STORAGE

The purpose of this section is to identify chemical feed and storage systems needed for the WHWTP.

The initial firm treatment capacity of the plant is 6.0 MGD. Future expansions will bring the total plant capacity to 9.0 MGD. Unless otherwise noted, where it is cost effective and appropriate for the overall design, the facilities are recommended to be sized to store chemicals at the initial average flow, which in this case is assumed to be 3.0 MGD. To minimize costs where practical, some chemical storage facilities are sized for the future average flow of 4.5 MGD. In either case, footprints of chemical areas are designed for future storage. Metering pumps have a lesser operating lifespan than storage facilities thus the metering pumps are sized for the maximum capacity at the initial plant design, 6.0 MGD. They can be replaced in the future expansion.

Table 8-1. Chemical Design Criteria

Operating Condition	Unit	Value
Initial Average Capacity	MGD	3.0
Initial Maximum Capacity	MGD	6.0
Future Average Capacity	MGD	4.5
Future Maximum Capacity	MGD	9.0

Based on the waste streams generated by the pretreatment and filter systems and the treatment of reclaimed return flow, some of the chemical calculations are based on a maximum influent capacity which is an additional 5 percent of the plant flow.

8.1 Pre-oxidation

Based on the information presented in Section 6 - Pretreatment, the use of sodium permanganate as a pre-oxidant is recommended. Storage and pumping equipment will be provided based on the use of a 20 percent solution. This provides the flexibility of using 40 percent solution in the future if demand increases or seasonal variations indicate the need for additional pre-oxidation. The design doses are presented in Table 8-2 based on the pilot test results. Initial analyses and discussions with Carus, a leading permanganate provider, indicated that the dose of sodium permanganate is approximately 20% greater than the dose of potassium permanganate to obtain equivalent water quality results.

Table 8-2. Sodium Permanganate Dose

Operating Condition	Dose (mg/L)
Minimum	0.48
Average	0.60
Maximum	0.72

The projected sodium permanganate consumption is shown in Table 8-3. A storage capacity of at least 30 days is recommended for sodium permanganate as well as all the chemicals on-site at average flow and average dose. This provides the operational flexibility to schedule delivery frequency based on usage depending on dosage and flow rates. It optimizes the storage capacity without providing too large of a tank which occupies floor space and also extends delivery scheduling. A capacity of 250 gallons would provide the 30 day storage at the initial average flow of 3.0 MGD, while 350 gallons would provide the 30 day storage for the future average flow of 4.5 MGD at buildout. Based on standard tote sizing, a 350 gallon storage tote is recommended.

Table 8-3. Estimated Sodium Permanganate Consumption at Average Flow

Operating Dose	Solution Consumption, gal/day Initial Average 3.0 MGD	Solution Consumption, gal/day Future Average 4.5 MGD
Minimum	5.2	7.8
Average	6.5	9.7
Maximum	7.8	11.7

8.2 pH Control

Both sulfuric acid and sodium hydroxide are required for pH control, as described in the following subsections.

8.2.1 Sulfuric Acid

During the pilot testing of the raw water, sulfuric acid was determined to be the optimum chemical for preliminary pH adjustment. Sulfuric acid will be injected upstream of the Actiflo® Carb unit for pH control. The pH of the raw water was adjusted during the pilot study to for optimization of the coagulation and organics removal process. A 93 percent sulfuric solution is recommended which has a specific gravity of 1.84. The recommended sulfuric acid dose is presented in Table 8-4.

Table 8-4. Sulfuric Acid Dose

Operating Condition	Initial pH	Target pH	Disinfection Dose (mg/L)
Minimum	8	6.5	40
Average	8	6.2	53
Maximum	8	6.0	70

Based on the recommended dose presented in Table 8-4, the projected sulfuric acid consumption is shown in Table 8-5. A capacity of 2,800 gallons would provide the 30 day storage at initial operation average capacity of 3.0 MGD. A capacity of 4,400 gallons would provide the 30 day storage at future (build out) average capacity of 4.5 MGD. Based on standard tank sizing, a 4,600 gallon storage tank is recommended. This capacity also meets the recommendation of storing sulfuric acid in a quantity of one truck load of chemical plus a minimum 25% to 50% reserve. A single truck will typically deliver 40,000 pounds, or 2,600 gallons, of sulfuric acid.

Table 8-5. Estimated Sulfuric Acid Consumption at Average Flow

Operating Dose	Solution Consumption, gal/day Initial Average 3.0 MGD	Solution Consumption, gal/day Future Average 4.5 MGD
Minimum	46	98
Average	93	146
Maximum	186	293

The most common material for storage of 70% or greater concentration sulfuric acid is carbon steel. Carbon steel is relatively inexpensive and offers good corrosion protection. A steel tank can also be constructed to ASME code to withstand pressure surges, as can inadvertently or accidentally happen from a tanker truck uncontrolled blow-off or air surge. When sulfuric acid first contacts the steel, iron sulfate (FeSO4) is produced. The iron sulfate coats the steel and forms a passivation film which protects the carbon steel from further corrosion. Coatings can be used to slow or eliminate corrosion of steel tanks, and reduce contamination of the acid, specifically from iron. Coating options include baked on phenolic coatings, which are relatively expensive since the entire tank must be placed inside a large oven to cure the coating. Glass lining is even more expensive, and used where the acid is extremely pure and no contamination is permitted. Hard rubber lining is also more expensive than baked phenolics, and is typically used in acid tanker transport cars.

Since iron is being added to the water as part of the coagulation filtration process anyway, iron contamination is not a critical factor. However, moisture can enter the tank during venting, and the resultant dilution of acid on top can form a “corrosion ring” inside the tank. A baked phenolic coating will add approximately 30% to 50% to the price of the steel tank. Large tanks may be too large to fit in most ovens for a baked phenolic coating, making it cost prohibitive.

Alternatively, an exterior acid resistant epoxy coating could be applied to the outside of the tank for aesthetics and corrosion protection in the event of an acid release.

Anodic protection (AP) is effective in minimizing tank corrosion. Anodic protection works by applying a current to the tank to increase the electrochemical potential. A more detailed analysis of AP versus a baked phenolic coating will be conducted in design to determine the best corrosion protection system for the tank.

High density linear polyethylene (HDLPE) and high density cross linked polyethylene (HDXLPE) are additional options for 93% sulfuric acid storage. Since sulfuric acid is about twice as heavy as water and puts a significant strain on a tank, HDPE is generally only recommended for sulfuric acid tanks less than 4,500 gallons. In addition, plastic tanks are subject to stress cracking. The potential for cracking increases with temperature, storage time, and acid strength. While there are many examples of HDPE sulfuric acid tanks with 30 or more years of service in excellent condition, the design life is twice as long for a steel tank with a comparable cost.

8.2.2 Sodium Hydroxide

Sodium hydroxide (caustic) will be injected upstream of the filters and clearwell for pH control. Sodium hydroxide can be supplied in solution form in up to a 50 percent concentration. Sodium hydroxide can also be purchased at 25 percent concentration.

When the temperature decreases to temperatures less than 55^{o}F, the higher concentration solution is more likely to crystallize and cause problems in the feed system. Typically, a heating system is installed to maintain a solution temperature above 55^{o}F. The average daily low temperature in Hollister, CA is below 55^{o}F the entire year, with the minimum occurring in December at 38^{o}F. The greatest potential for freezing is not in the storage tank, but in the process piping from the tank to the injection location. However, both the tank and process piping will need heat tracing to prevent freezing.

Sodium hydroxide at 25 percent is less likely to crystallize, or freeze, at cooler temperatures. However, based on the dosage rates presented in Table 8-6, a significant storage facility would be required. A 50 percent solution is recommended to reduce the overall storage requirements and footprint.

The doses were determined using the raw water quality data assuming adjusted pH and alkalinity from prior acid addition. The target finished water pH of 8.0 was used to calculate the required sodium hydroxide dose.

Table 8-6. Sodium Hydroxide Dose

Operating Condition	Dose (mg/L)
Minimum	28
Average	40
Maximum	50

Based on the recommended dosage of 50 percent solution, the anticipated sodium hydroxide consumption is in Table 8-7. For initial operation, a minimum storage of 5,220 gallons is recommended based on the usage of a 50 percent solution and will provide storage for approximately 30 days at average flow and average dose. The actual usage will vary based on raw water quality and demand. For future buildout, a minimum total storage of 8,000 gallons is recommended based on the usage of a 50 percent solution. Based on the incremental storage volume required at buildout, an 8,000 gallon tank is recommended for the initial construction.

Table 8-7. Estimated Sodium Hydroxide Consumption at Average Flow

Operating Dose	Initial Average 3.0 MGD		Future (Build-out) Average 4.5 MGD	
	25% Solution Consumption, gal/day	50% Solution Consumption, gal/day	25% Solution Consumption, gal/day	50% Solution Consumption, gal/day
Minimum	174	87	348	174
Average	348	174	522	261
Maximum	696	348	1,044	522

Two metering pumps (1 duty, 1 standby) will be used to supply sodium hydroxide for pretreatment.

8.3 Coagulants

Coagulants will be dosed in three locations: into the Actiflo® Carb unit, downstream of the Actiflo® Carb unit effluent, and upstream of the backwash waste plate settler. The reason for injection downstream of the Actiflo® Carb unit is to react with any potential polymer carryover. Coagulants typically used for these applications include aluminum sulfate (alum), ferric chloride, and cationic polymers. The advantages and disadvantages of these coagulants are summarized in Table 8-8.

Table 8-8. Comparison of Coagulant Alternatives

Coagulant	Advantages	Disadvantages
Alum	Lowest chemical cost Readily available Effective for most surface waters over a wide pH range (5.5 to 7.5)	Consumes alkalinity Not always effective for TOC and color removal
Ferric Chloride	Effective for most surface waters over wide pH range (5.0 to 8.5) Normally provides better Total Organic Carbon (TOC) removal than aluminum based coagulants	Consumes alkalinity Slightly higher cost than alum Corrosive to many materials, special design considerations for storage areas Causes staining of surfaces it comes in contact with
Cationic Polymer	Effective for removals of fine or colloidal solids Produces less sludge than alum or ferric coagulants. Works best as coagulant aid instead of primary coagulant	Highest cost per pound Not effective in TOC removal

Based on the bench scale and pilot testing, the preferred coagulant is ferric chloride. The ferric chloride feed system into the Actiflo® Carb unit will be provided as part of the Kruger Actiflo® Carb system contract. However, storage of the coagulant will be sized by HDR and provided by the Contractor. A second individual feed unit will be provided for the backwash waste process, though the storage will be combined.

Bench and pilot testing indicated that alum could also be an effective coagulant for use in this treatment process and does not contain trace levels of manganese that require supplemental removal as does the ferric. Design for and initial implementation of ferric chloride allows for the potential future coagulant switch to alum, if determined to be favorable based on future supplemental bench testing.

Table 8-9. Coagulant (Ferric Chloride) Doses

Operating Condition	Actiflo® Carb Unit Dose mg/L	Post-Actiflo® Carb Unit Dose mg/L	Backwash Waste Dose mg/L
Minimum	10 mg/L	0.75 mg/L	8 mg/L
Average	20 mg/L	1.0 mg/L	12 mg/L
Maximum	30 mg/L	1.25 mg/L	16 mg/L

Table 8-10 presents the anticipated ferric chloride use and recommended supply based on the doses assumed in Table 8-9. The overall flow for backwash processes is limited to 10% of the overall plant flow. At the initial operation average daily flow of 3.0 MGD, the average ferric chloride use will be approximately 105/5.2/4.7 gallons per day (gal/day) for the pretreatment, post treatment, and backwash, respectively. Based on manufacturer standard sizes, a 3,650-gallon storage tank provides approximately 33 days of storage at average use. At the future buildout average daily flow of 4.5 MGD, the average ferric chloride use will be approximately 165/7.9/4.7 gallons per day (gal/day) for the pretreatment, post-treatment, and backwash, respectively. Total storage requirements would be 5,100 gallons. To ensure sufficient storage volume for acceptance of full truck deliveries (typically 3,000 to 5,000 gallons) and to maximize the cost effectiveness for future buildout, a 6,500 gallon tank (which includes approximately 25% reserve volume) is recommended for initial installation.

Table 8-10. Estimated Ferric Chloride Use at Average Flow

Operating Dose	Initial Average 3.0 MGD			Future (Build-out) Average 4.5 MGD		
	Pretreatment Usage, gal/day	Post-Actiflo® Carb Unit Dose, gal/day	Backwash Waste Usage, gal/day	Pretreatment Usage, gal/day	Post-Actiflo® Carb Unit Dose, gal/day	Backwash Waste Usage, gal/day
Minimum	52	3.9	3.2	83	5.9	3.2
Average	105	5.2	4.7	165	7.9	4.7
Maximum	157	6.6	6.3	248	9.8	6.3

Two metering pumps (1 duty, 1 standby) will be used to supply ferric chloride for pretreatment. In addition, two other metering pumps (1 duty, 1 standby) will be provided to handle the low volume injection of ferric chloride into the backwash recovery system.

8.4 Disinfection

Sodium hypochlorite will be used at the water treatment plant for the following:

- Maintain a chlorine residual through the filter, for improved manganese removal.
- Disinfectant to achieve required CT in the treated water storage tank and a chlorine residual in the distribution system.

This Section summarizes the requirements of the sodium hypochlorite storage and feed system needed for the WHWTP.

The projected dosage rates are shown in Table 8-11. These doses will vary depending on the water source and season.

Table 8-11. Sodium Hypochlorite Disinfection Dose

Operating Condition	Pre-Filter Dose (mg/L)	TW Storage Tank Dose (mg/L)
Minimum	1.0	1.5
Average	1.25	3.0
Maximum	2.0	4.5

Sodium hypochlorite is subject to degradation over time. The primary degradation products are chlorate and chlorite, which are a health concern in drinking water. The rate of degradation increases with hypochlorite concentration, storage temperature, and the presence of light. Several methods are used to manage hypochlorite during storage. They include use of immersed coolers in the storage tank, cooling the chemical storage area, and providing additional storage capacity to allow for on-site dilution.

Together, with all the bulk chemicals, the hypochlorite storage tank will be located outside. The hypochorite in tank will be insulated and covered with a canopy to decrease degradation due to UV exposure. Table 8-12 presents the anticipated sodium hypochlorite consumption based on 12.5 percent sodium hypochlorite solution, initial and future average flow rates, and the doses given in Table 8-11. At the initial average daily flow of 3.0 MGD, the average sodium hypochlorite for consumption will be approximately 108 gal/day. At the future average daily flow of 4.5 MGD, the average sodium hypochlorite for pretreatment and disinfection will be approximately 168 gal/day.

Table 8-12. Estimated Sodium Hypochlorite Consumption

Operating Dose	Initial Average 3.0 MGD		Future (Build-out) Average 4.5 MGD	
	Pre-Filter Consumption, (gal/day)	Pre-Storage Tank Consumption, (gal/day)	Consumption, (gal/day)	Consumption, (gal/day)
Minimum	24	36	40	54
Average	36	72	60	108
Maximum	48	108	79	162

Based on initial average conditions, a 3,900-gallon tank would provide 32 days of storage. Upon expansion, a second tank would be added to maintain 30 days of storage at the higher average flow.

Two pumps will be used to supply sodium hypochlorite for pretreatment and post treatment disinfection. One pump will be dedicated to each. A third pump will be installed as a common standby.

Figure 8-1. Sodium Hypochlorite Tank, Pumps, and Controls

The tank material shall be linear high-density polyethylene (HDPE) and the piping material shall be schedule-80 polyvinyl chloride (PVC). Sodium hypochlorite solution will degrade and degas during storage, which can cause vapor lock in some diaphragm metering pumps that are not designed to handle off-gassing liquids. While this limits the selection of diaphragm metering pumps, it does not eliminate them. Alternatively, peristaltic metering pumps can be used as they are not subject to vapor lock.

8.5 Process Chemicals

Additional chemicals that optimize the treatment process include coagulant polymer and powdered activated carbon, as discussed in the following subsections.

8.5.1 Coagulant Polymer

An anionic polymer will be dosed to the Actiflo® Carb unit and the backwash recovery system to assist in coagulation and enhance performance. Using polymer will lower the coagulant dosage and increase operational flexibility of the reclaim water system. Care must be taken to not add excessive polymer that could carry over and foul other processes. The pilot study used Hydrex 3502, an anionic dry product of Crown Solutions. The polymer was made up as a 0.10% solution (1 gram of polymer for every 1 liter of water).

The usage in the backwash waste recovery process will be significantly less than the feed into the Actiflo®Carb unit, due to the lower flow rates. The waste stream will be limited to 10 percent of the overall plant flow per regulations with reclaim. Table 8-13 presents the range of polymer doses expected for use in the Actiflo® Carb unit (as determined by the polymer optimization of the pilot study) and in the backwash recovery stream.

Table 8-13. Polymer Doses

Operating Condition	Actiflo®Carb Unit Dose, mg/L	Backwash Waste Dose, mg/L
Minimum	0.70	0.40
Average	0.90	0.50
Maximum	1.00	0.60

Table 8-14 presents the anticipated polymer consumption. Consumption is based upon a solution diluted to 0.1% net active polymer. Polymer can be stored as 30% solution and diluted to 0.1% net active polymer. The 30 percent polymer can be delivered and stored in 55-gallon drums or a tote, depending on usage. One drum would store chemicals for approximately 6 days at average use so a tote is recommended. A mixer will be needed to keep the polymer blended while stored.

Table 8-14. Estimated 30% Solution Polymer Consumption

Operating Dose	Initial Average (3 MGD) Actiflo®Carb, gal/day	Future (Buildout) Average (4.5 MGD) Actiflo®Carb, gal/day	Initial Average (0.6 MGD) Backwash Waste, gal/day	Future (Buildout) Average (0.9 MGD) Backwash Waste, gal/day
Minimum	5.84	9.63	0.42	0.63
Average	7.51	12.38	0.63	0.95
Maximum	8.34	13.76	0.75	1.13

Polymer solution will be made in batches using a polymer blending unit provided. As part of the Kruger Actiflo®Carb system contract the unit will include a polymer pump to blend polymer with dilution water. The polymer flow and dilution water flow are paced to give a pre-selected dilution rate. The batches will be stored in a day tank and metered to the injection points.

Figure 8-2. Polymer Storage and Feed System

Since two processes will utilize polymer, two metering systems will be required. However, both systems can draw from a shared polymer storage tank. Based on the average flows for each stream, a 400-gallon tote is recommended which will provide 30 days of storage.

8.5.2 Powdered Activated Carbon

Powdered activated carbon (PAC) will be injected directly into the Actiflo® Carb unit. Two storage and feed system options include a skid mounted storage hopper feed system and a silo feed system.

8.5.2.1 Storage Hopper Feed System

For systems with periodic use, PAC can be provided in storage sacks in various sizes ranging from 50 lb to 1,000 lb super sacks. The sack size provided has a direct impact on the maintenance and equipment requirements. Use of the larger sack would require an onsite forklift and hoist for loading the chemical along with a larger hopper for storage. However, the larger sack would require less frequent loading than the smaller sack. Based on the feed rates, one super sack (1,000 lbs) would be loaded and used every two days during initial operation and more frequently during future operation.

The use of the storage hopper feed system requires the following:

- Means of unloading the large bags of chemical with a forklift or other equipment.
- Space for storing several super sacks of PAC.
- Space adjacent to the feed system to allow maneuvering of the bags to place in the feeder and for storing empty bags for supplier pickup.
- Approximately 18 to 19 feet of headroom to allow hoisting of the super sacks for placement in the hopper.

The hopper feeds the dry PAC into a mix tank for blending with water and creation of a PAC slurry. The PAC storage and feed system includes a storage hopper, volumetric screw feeder, trolley and hoist system, dust collector, and bulk bag unloader. The operator would be required to move the PAC super sack to the bulk bag unloader and then lift the sack above the feed system into the hopper. The projected feed rates as shown in Table 8-15 exceeds that which is typically used in sack feeders. Based on the continuous dosing requirement for PAC at the WHWTP, the operation intensity, the untidy and dusty nature of the feed system, the general dissatisfaction experienced by operators of similar systems, the bulk bag hopper feed system is not recommended.

8.5.2.2 Silo Feed System

A silo feed system requires reduced maintenance and operator attention as compared to the sack loading system. The silo stores approximately 25 tons of PAC. The size is based on the typical delivery size of 20 tons. When receiving a PAC load, the chemical supplier charges based on the truckload, not necessarily the quantity of chemical required. Therefore, storage capacity is recommended to exceed a typical delivery load. Dry PAC is be pneumatically unloaded from a bulk truck transport trailer. Utilizing the truck's compressor and hose connected to an installed carbon steel pipeline, the PAC is transported and discharged tangentially into a welded carbon steel PAC storage silo.

The silo would be located outdoors and include two feed systems (1 duty and 1 standby) at the base with a mix tank and pumping equipment. At the base of the silo unit, there is a conical discharge and a PAC feed train consisting of valves, volumetric feeder, wetting bowl, hydraulic eductor, heater, and exhaust fan. The carbon slurry from the eductor is transported through pneumatically actuated valves to the injection point. A pneumatic dust collector helps to eliminate concerns during delivery and operation. A silo feed system similar to that shown in Figure 8-3 is recommended for the WHWTP. The silo's site location will be selected to minimize visual impacts from the surrounding area.

Figure 8-3. Silo Feed System

The feed rates and dosages for the PAC system are shown in Table 8-15.

Table 8-15. Projected PAC Usage

Operating Conditions	Dose (mg/L)	PAC Usage at Average Flow	
		Initial (3.0 MGD) (lb/d)	Future (4.5 MGD) (lb/day)
Minimum	10	275	413
Average	20	550	826
Maximum	30	825	1,238

8.6 General Design Criteria

The slimy characteristics of polymer make it messy to store and handle. The floor in polymer storage and blending areas may become slick and is a slip/fall hazard. Much water is required to wash down these areas amidst the interference created by the piping and equipment. To improve housekeeping and safety, a concrete curb will enclose the storage and blending area to confine the polymer in the case of a spill. The enclosed area will have a hose bib with cold and warm water to permit wash downs and a floor drain connected to the sewer. A gnarled, grated floor suspended 6 to 12 inches above the slab will be installed alleviate slip/fall events.

8.6.1 Floor Plan

A full code plan will be prepared during design to determine chemical compatibility. The chemicals will be stored outdoors. Sufficient secondary containment storage will be provided for chemical spills and rain events.

8.6.2 Chemical Storage Tanks

High density polyethylene or fiberglass reinforced plastic tanks will be used for liquid chemical storage, except for the sulfuric acid which will be stored in a carbon steel tank. For sodium hypochlorite, which is subject to degradation, an insulated UV resistant tank will be used. The tanks are sized to meet supply requirements as well as to accommodate bulk delivery when appropriate. Sodium hypochlorite has a limited shelf life; manufacturers of other chemicals recommend a maximum shelf life of 60 to 90 days.

8.6.3 Chemical Metering Pumps

Chemical feed pumps shall be diaphragm pumps with digital, step-less motors or equivalent. All continuous use chemical pumps shall have at least one pump in standby. One spare pump for non-continuous applications shall be stored on the shelf. The pump and all appurtenances shall be constructed of corrosion-resistant materials suitable for the chemical handled. The chemical pumps shall be installed in an area within the containment area to capture any leaks.

8.6.4 Chemical Pipelines

Double-wall piping will be used for chemical lines outside of containment areas. PVC and CPVC pipe will be used for containment pipe and tubing will be used to carry the product. Leak detectors will be provided at the low point on each chemical line. The leak detection panel will feed an alarm to the main plant computer. Flushing taps will be provided on all chemical pipes to allow for draining or flushing of chemicals. Flushing taps will be provided between the tank and metering pumps, and the metering pump and point of use.

Figure 8-4. Example Chemical Metering Pump Gallery

8.6.5 Chemical Containment

All chemicals stored in tanks will have either a fill station to receive bulk delivery or means to access the tank in order to fill it directly. A sump or curbed containment area will be provided for each chemical sufficient to hold the contents of one tank plus 20 minutes of fire sprinkler flow. In case of a large leak, a vacuum truck will be called in to clean the leakage. Alternatively, a sump pump can be used to recover spillage. Each containment area will be provided with an eye wash station and additional stations will be located outside the storage rooms. Storage closets for safety equipment shall be provided near the containment areas.

The floor around the permanent tanks will be depressed or the tanks will be elevated around a barrier wall for spill containment. The containment area will be sized to contain the largest tank volume at build-out conditions plus 20 minutes of fire sprinkler flow. A grated floor will span the containment area around the tanks. The metering pumps will be placed atop concrete pedestals that rise above the grating in the containment area. Permanent tanks will sit atop concrete pedestals in order to ensure the pumps maintain a flooded suction. The containment area for each chemical will be separated to avoid cross contamination. All containment areas will be lined with a coating system that is compatible with the stored chemicals to protect the concrete. Chemicals spilled into the containment areas will be pumped out.

Figure 8-5. Chemical System and Concrete Containment Area

8.6.6 Seismic Restraint

Chemical storage area and containment structures shall be designed in accordance with local seismic requirements. Chemical storage tanks shall be provided with a seismic restraint system to resist seismic loading when the tank is full. Tank restraints shall be designed and stamped by an engineer registered in the State of California.

8.7 Design Summary

Table 8-16 summarizes the preliminary design criteria of the chemical systems.

Table 8-16. Chemical System Design Summary

Chemical	Design Parameter	Design Criteria (at 3 MGD)	Design Criteria (at 4.5 MGD)
Sodium Permanganate	Number of Storage Totes	1	1
	Storage Tank Volume, each	250 gallons	350 gallons
	Storage Tank Capacity at Average Dosage	39 days	36 days
	Number of Pumps	2, including one spare	2, including one spare
	Pump Capacity, each	0.54 gal/hr	0.81 gal/hr
Sulfuric Acid	Number of Storage Tanks	1	1
	Storage Tank Volume, each	4,600 gallons	4,600 gallons
	Storage Tank Capacity at Average Dosage	50 days	31 days
	Number of Pumps	2, including one spare	2, including one spare
	Pump Capacity, each	7.7 gal/hr	12.2 gal/hr

Chemical	Design Parameter		Design Criteria (at 3 MGD)	Design Criteria (at 4.5 MGD)
Sodium Hydroxide	Number of Storage Tanks		1	1
	Storage Tank Volume, each		8,000 gallons	8,000 gallons
	Storage Tank Capacity at Average Dosage		46 days	31 days
	Number of Pumps		2, including one spare	2, including one spare
	Pump Capacity, each		14.5 gal/hr	21.7 gal/hr
Ferric Chloride	Number of Storage Tanks		1	1
	Storage Tank Volume, each		3,650 gallons	6,500 gallons
	Storage Tank Capacity at Average Dosage		33 days	30 days
	Number of Large Pumps		2, including one spare	2, including one spare
	Large Pump Capacity, each		8.7 gal/hr	13.8 gal/hr
	Number of Small Pumps		2, including one spare	2, including one spare
	Small Pump Capacity, each		0.4 gal/hr	0.4 gal/hr
Sodium Hypochlorite	Number of Storage Tanks		1	2
	Storage Tank Volume, each		3,900 gallons	3,000 gallons
	Storage Tank Capacity Average Dosage		32 days	30 days
	Disinfection Pumps (2 pumps dedicated for pre/post treatment, 1 common spare)	Number of Pumps	3 (2 duty, 1 spare)	3 (2 duty, 1 spare)
		Pump Capacity, each	6.0 gal/hr	9.0 gal/hr
Coagulant Polymer	Number of Storage Tanks		1	1
	Storage Tank Volume, each		300 gallons	400 gallons
	Storage Tank Capacity at Average Dosage		31 days	31 days
	Polymer Blend Units		1	1
	Diluted Polymer Batch Tank Capacity		250 gal	300 gal
	Number of Pumps		3 (2 duty each feed, 1 common standby)	3 (2 duty each feed, 1 common standby)
Powdered Activated Carbon (PAC)	Storage Capacity		50,000 lb	50,000 lb
	Silo Diameter		14 ft	14 ft
	Silo Height		38 ft	38 ft
	Usage at Average Dose		550 lb/day	826 lb/day

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9 Treated Water Storage Tank and Pipeline

The purpose of this section is to evaluate the size and material of the proposed storage tank at the WHWTP to ensure sufficient storage capacity and disinfection contact time of the treated water prior to distribution.

9.1 Storage Tank Design Criteria and Sizing

The initial 6.0 MGD treatment capacity, treated water contact time (CT), plant water needs, and overall system demand indicate that 530,000 gallons are initially required for clearwell storage at the WHWTP. Expansion of the WHWTP to 9.0 MGD in the future will require an additional 200,000 gallons of storage. To address supplemental system-wide storage capacity needs, as referenced in the Hollister Urban Area Water and Wastewater Master Plan, the future storage capacity at the WHWTP site could also include construction of a new 2.0 MG tank in addition to the initial storage required. The future storage tank can be sized to accommodate the future treatment capacity along with the 2.0 MG system storage. The actual usage and projections can be used in the future to reassess the storage requirements. It is important that the footprint of the second tank be considered along with the water levels in the tanks when laying out the site plan. The storage tank capacity requirements are included in Table 9-1.

Table 9-1. Storage Tank Capacity Criteria

Storage	Required Capacity (gal)	
	Initial (6 MGD)	Future (9 MGD)
Peak System Water Demand [a]	250,000	375,000
Filter Backwash Demand [b]	79,200	105,600
On-site Fire Supply [c]	96,000	96,000
CT Disinfection	107,143	160,714
Recommended Storage	532,330	737,314

Notes:

a) Peak hourly demands for a typical WTP are 1.5 to 2 times the maximum day demand. However, the design of the WHWTP assumes that peak demands greater than 6 MGD will be supplied from within the system by groundwater wells. Assumes that the plant can return to service within 2 hours of shutdown at average treatment conditions or 1 hour of shutdown at maximum treatment conditions.

b) Amount of water required for backwash processes (for one filter) over a 24-hour period. Conventional concrete filters will backwash once daily.

c) 800 gpm per AWWA M31, Iowa State University Method. Assumes 5,000 square foot, 16 foot high building. All fire flows less than 2,500 gpm will be for a 2 hour duration. Additional fire protection will be provided by the distribution system.

The hydraulic grades and processes will be designed considering the high water level in a 0.55 MG tank. If a future 2.0 MG tank is required as demand increases, there are two options to synchronize operations with the existing 0.55 MG tank. The first option is to bury the new 2.0

MG tank to match water elevations with the existing tank. The second option will be to provide a small pump station to lift treated water to the higher surface elevation in the larger tank. This will require future analysis as the plant expands to meet increasing demands.

Conventional treatment plants typically receive 2.5 log credit towards the 3 log requirement for *Giardia* inactivation. The disinfection effectiveness is dependent on the filtered water's adjusted pH, temperature, chlorine residual, and contact time. The following assumptions have been made for sizing the tank:

- Minimum Temperature: 5˚ C
- Maximum Temperature: 20˚ C
- Maximum pH: 8.0
- Minimum Chlorine Residual: 1.0 mg/L
- CT required for 0.5 log *Giardia* inactivation: 36 mg/L-minutes

The actual contact time is a function of the tenth percentile of the residence time within the tank, or T_{10}/T. The baffling classifications for tanks are broken down into the conditions in Table 9-2.

Table 9-2. Baffling Classifications

Condition	T_{10}/T	Baffling Description
Unbaffled (mixed flow)	0.1	None, agitated basin, very low length to width ratio, high inlet and outlet flow velocities.
Poor	0.3	Single or multiple unbaffled inlets and outlets, no intra-basin baffles.
Average	0.5	Baffled inlet or outlet with some intra-basin baffles.
Superior	0.7	Perforated inlet baffle, serpentine or perforate, intra-basin baffles, outlet weir or perforated launders.
Perfect (plug flow)	1.0	Very high length to width ratio (pipeline flow), perforated inlet, outlet, and intra-basin baffles.

Notes:

a) *Surface Water Treatment Staff Guidance Manual*. CDPH, Office of Drinking Water. May 15, 1991.

HDR recommends an internal baffled storage tank with a baffled inlet and outlet that would have a sufficient length to width ratio to obtain a T_{10}/T value of 0.7. Based on the recommended configuration and the classifications presented in Table 9-2 a minimum of a 550,000-gallon storage tank is required for a maximum treatment capacity of 6.0 MGD.

9.2 Tank Material Evaluation

There are typically three potential options for tank construction: welded steel, glass lined and coated bolted steel, and prestressed concrete. However, due to the location of the tanks and the site terrain, the tank will need to be differentially buried, buried more on one site of the tank than on the other, creating different loading conditions on the tank and foundation. As the tank will be buried, concrete is the only viable alternative. Steel tanks can not be buried due to corrosion concerns and issues with loading against the tank walls. Since there is only one potential construction material, this section will discuss the potential construction methods of concrete tanks.

9.2.1 Prestressed Concrete Tank Construction Methods

Prestressed concrete (as opposed to conventional reinforced concrete) is a cost effective and durable form of construction for potable water tanks. Per AWWA D110, prestressed concrete is defined as concrete in which internal compressive stresses of such magnitude and distribution are introduced that the tensile stresses resulting from the service loads are counteracted to the desired degree. Circumferential prestressing of the tank wall is introduced herein by the helical application of high-strength steel wire or strand under controlled tension and vertical prestressing by post-tensioned, high strength steel bar or strand tendons. The prestressing of concrete combines the strength of concrete in axial compression with the strength of steel in axial tension. The benefit of this type of construction is that it prevents the cracking of concrete and thereby minimizes potential leakage. Prestressed concrete tanks are typically constructed utilizing a “machine-wrapped circumferential prestressing” technique. These tanks are constructed on site using tilt-up panels and/or cast-in-place techniques by the manufacturer.

As a general rule, prestressed concrete tanks become more cost effective as the size of the tank increases. Another feature of prestressed tank is that they can be constructed with ‘architectural treatments’, thus adding aesthetic features. The aesthetics of prestressed concrete tanks are a major advantage in residential areas. The tanks do not require painting or coating, thus minimizing ongoing maintenance requirements and costs. These tanks can be constructed with concrete roofs (flat or domed) or with aluminum domed roofs.

Table 9-3. Features of a Prestressed Concrete Tank

Item	Feature/Options
Roof	Flat column supported, domed concrete, domed aluminum, or column supported aluminum.
Floor	Concrete.
Coating System	None.
Aesthetics/Colors	Architectural treatments are available to help blend the tank into the surrounding environment.

There are five types of core walls that are a part of the prestressed concrete tank construction:

- AWWA D110 Type I - Cast-in-Place Concrete with Vertical Prestressed Reinforcement
- AWWA D110 Type II - Shotcrete with a Steel Diaphragm
- AWWA D110 Type III - Precast Concrete with a Steel Diaphragm
- AWWA D110 Type IV - Cast-in-Place with a Steel Diaphragm

- AWWA D115 – Tendon-Prestressed Concrete

AWWA D110 Type I, and AWWA D110 Type III tanks have been deemed relevant for this project and will be evaluated as part of this Section. AWWA D110 Types II and IV, and AWWA D115 tanks are not suitable for this project due to a lack of tank manufacturers that would use these systems.

9.2.2 Tank Procurement Considerations

Normally, the prestressed concrete tanks are procured in a manner similar to steel tanks, which involves the tank supplier both designing and erecting the tank under one single source contract. In some cases the prestressed concrete tanks have been procured by other methods as follows:

1. Tank designed by Engineer and constructed by the tank supplier.
2. Tank designed by Engineer and concrete work done by third party Contractor, with prestressed wrapping (or tensioning of tendons) done by Tank Supplier.
3. Tank designed by Tank Supplier and concrete work done by third party Contractor, with prestressed wrapping (or tensioning of tendons) done by Tank Supplier.

Items 1 through 3 have certain advantages for ensuring the tank is designed the way the Owner prefers or to enhance competition for concrete work. The significant drawback of these procurement methods is that they rely on multiple party responsibilities. In the event of a problem with the tank, each of the parties may try to blame the other to avoid taking responsibility for fixing the problem. Single source responsibility is the preferred method to reduce risk of unforeseen costs to the Owner.

9.2.3 AWWA D110 – Wire- and Strand-Wound, Circular, Prestressed Concrete Water Tanks

AWWA D110 encompasses four types of concrete tanks. Of these, two types, Type I-cast-in-place concrete with vertical prestressed reinforcement and Type III-precast concrete with a steel diaphragm, show promise for this specific project. Both types accomplish horizontal reinforcement of the tank wall by wrapping wire around the tank perimeter. The essential difference is that Type I has cast-in-place walls, and Type III has precast walls. Because of this, there are profound differences in construction between the two types. Of particular importance in high seismic zones, is that Type I integrates the seismic restraint connection between floor and walls by casting cables into the floor and then into the walls. Type III walls are precast, and the seismic restraint connection is done secondarily and is not directly cast into the walls. A second major difference is the method that is used to make the tank watertight. For Type I tanks, a water stop is cast into the concrete joints. For the Type III tank, the steel plates are bonded to the diaphragms between individual wall panels, and at the floor-to-wall connection, an exposed water stop is used. Details for each construction method are included below.

In recent years, two companies have performed the majority of concrete tank construction conforming to AWWA D110 Type I and Type III, DYK and Natgun, respectively. In 2011, these companies merged into a single entity, DN Tanks. For purposes of organization and ease of reference, this report will continue to refer to Type I tanks as constructed by DYK and Type III tanks as constructed by Natgun.

9.2.3.1 Type I Construction – Cast-in-Place Concrete with Vertical Prestressed Reinforcement

An AWWA D110 Type I tank uses cast-in-place concrete. It is the only AWWA D110 tank that does not use a steel diaphragm. Instead it employs high strength vertical steel tendons conforming to ACI 318 and ASTM A416 or ASTM A722. After the concrete for the reinforced tank floor and foundation is poured, forms for the tank walls are constructed with the vertical tendons anchored inside of the wall forms. The concrete walls are specified to be minimum 8-inches thick and anchored to the foundation with flexible connections that reduce the bending moment, thus increasing the seismic performance capabilities of the tank. A similar connection is made between the tank walls and the tank roof. Embedded PVC water stops and bearing pads are used at the base of the walls to prevent leakage. Concrete for the walls is poured from the top of the walls in between the forms and vibration equipment is used to eliminate voids as the concrete is placed in lifts from bottom to top.

Figure 9-1. Partially Buried Tank Construction 6.0 MG Type I Tank (Yucaipa, CA)

The remaining tank construction processes are similar for both Type I and Type III construction. Once the concrete cures an automated prestressing machine is used to wrap the exterior of the tank at the specified pressure. To protect the steel reinforcement from corrosion layers of shotcrete are applied to the tank exterior. Applying it in multiple layers allows the shotcrete a more uniform cure.

Figure 9-2. DYK Construction of Two Partially Buried 1.4 MG Type I Tanks (Sacramento County, CA)

Figure 9-3. DYK Prestressing Wire Strand Placement Detail Prior to Shotcrete (Sacramento County, CA)

Currently, DYK is the only constructor of Type I tanks and they have an extensive project list throughout the United States, including high seismic areas. The proposed unbalanced loading may create a possibility of sliding that will require additional considerations by DYK in the tank design.

Table 9-4. Pros and Cons of AWWA D110 Type I Construction

Pros	Cons
Extensive seismic experience in the western U.S.	Limited competition.
Proven robust cast-in-place design	Requires large machine for constructing pretensioning
Good performance of tanks in actual seismic events	Requires all outdoor concrete pours during short construction season.
Cost competitive with Type III construction	

9.2.3.2 Type III Construction – Precast Concrete with a Steel Diaphragm

Competition for construction of a Type III tank is limited in the west region. There are two manufacturers who have the history of Type III construction in the eastern and central United States: Natgun and Preload.

Type III tanks are built using precast concrete panels with embedded steel diaphragm to provide a watertight tank. The steel diaphragm provides the formwork for the concrete. Per AWWA D110, diaphragms are specified to be minimum 26 gauge and be continuous for the entire height of the tank and the tank walls are specified to be minimum 4-inches thick.

As the panels are tilted up in to place they are secured to the tank floor with a flexible connection to the tank floor for the transfer of seismic loads to the tank foundation.

Figure 9-4. Natgun Type III Construction - Tilting Up Precast Walls (Courtesy Natgun Corporation)

The panels are set on a bearing pad on the tank foundation. On the interior of the tank the PVC waterstop that was partially embedded in the tank foundation is connected to the tank walls with reinforced concrete. To complete the flexible connection the remainder of the waterstop is covered in shotcrete and a sponge filler pad is used to prevent the bonding of the shotcrete to the tank floor. There is no reinforcement that connects the tank floor to the wall. This construction process, including the waterstop, bearing pad, and sponger filler pad effectively creates a watertight seal and a flexible connection at the base of the tank wall. This construction method is shown below in Figure 9-5 per drawings provided by Natgun.

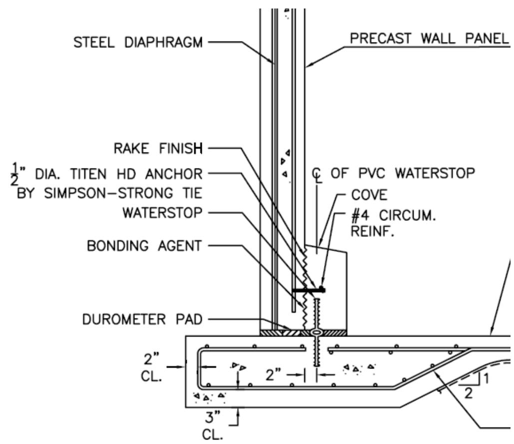

Figure 9-5. Type III Wall-Floor Connection (© Natgun Corporation)

The tank panels are typically placed about 7 inches apart and the gap is closed with steel plates that are bonded to the steel diaphragm with a polysulfide sealant. A high strength mortar is then used between the gaps to create a continuous surface. Once the tank shell has been constructed the tank manufacturer uses shotcrete to cover the diaphragm to protect from corrosion and in preparation for prestressing.

Steel wire is then wrapped around the cylinder with a stressing machine to compress the tank at a specified pressure. The size and specific design criteria of the wire are specified to conform to AWWA D110 standards while the spacing of the wire is determined by the tank designer. After the tank has been wrapped a second layer of shotcrete is applied to fully encase the prestress wire and provide additional corrosion protection.

In considering Type III construction, the resumes of the manufacturers must be considered. Both Natgun and DYK are experienced in storage tank design in highly seismic areas of California. Currently, Natgun has 18 concrete tanks built or under construction in seismic zones 3 and 4. Hollister is in Zone 4, located in one of the greatest seismic zones in California near the San Andreas Fault. DYK does design and construct Type III tanks, however their experience and design specialty is in Type I construction. Preload has been constructing concrete storage tanks since 1930. However, their experience lies mostly in the Midwest and eastern portions of the United States which do not have the seismic conditions that will be experienced in the western United States.

Figure 9-6. Prestress Wire Tank Wrapping (Courtesy Natgun Corporation)

Table 9-5. Pros and Cons of AWWA D110 Type III Construction

Pros	Cons
Ability to mobilize and begin construction early by starting precasting of panels off-site.	Less overall experience in seismic areas compared to Type 1.
Moderate level of seismic experience.	Requires space on site for efficient construction of precast walls.

9.3 Storage Tank Conclusion and Recommendations

HDR has consulted DN Tanks for specific information on this project. For tanks of this size, less than 1.0 MG, Type III construction is the most economical option. As the tank size increases, the costs become more competitive until reaching a breakpoint where Type I becomes more economical, typically in the 4.0 to 5.0 MG range. For the construction of both the 0.55 MG and 2.0 MG tanks, a Type III presents the most benefits to the MOU Parties.

A standard 0.55 MG Type III tank with a domed roof has an estimated cost of $575,000. This assumes a 54 foot inside diameter and a 32 foot side water depth with 3 feet of freeboard. In some instances the domed roof presents issues with aesthetics and visibility. Given the diameter of the tank and a typical 4:1 roof slope, the roof will not extend significantly above the sidewall of the tank. At this diameter, the center of the tank dome would be approximately 7-feet above the sidewall elevation. However, if this presents concerns, a tank with a column supported flat roof can be constructed for a total estimated cost of $630,000. The costs assume that the tank will be partially and differentially buried, but do not include deep foundations that may be required pending a soils investigation. Note that if the differential burial exceeds 5 feet between sides, an extended footing may be required which would increase the costs by approximately $20,000 to $40,000. The costs also do not include appurtenances such as ladders, handrails, and access hatches.

Given the future projections for storage capabilities, the site should be optimized to allow the construction of two 0.55 MG tanks (one initial and one future with the dimensions described above) and one – 2.0 MG tank with an inside diameter of 139 feet. Depending on the location selected and associated elevation, the sidewall height can be adjusted to achieve the same water elevations in the tanks.

Based on the preliminary design criteria a 0.55 MG prestressed concrete tank would meet the CT and storage requirements of the treatment facility. An AWWA D110 Type III tank is recommended for cost savings, however a Type I tank could be constructed and more information could be provided if needed.

Table 9-6. Preliminary Storage Tank Design Criteria

Item	Unit	Value
Material	-	Prestressed Concrete, AWWA D110, Type III
Capacity	MG	0.55
Diameter	ft	54
High Water Level	ft	32
Freeboard	ft	3
Sidewall Depth	ft	35
Roof Type	-	Concrete Dome
Estimated Cost[(a)]	$	$575,000 - $630,000
	Moderate level of seismic experience.	Requires space on site for efficient construction of precast walls.

Notes:
a) Cost range includes potential addition of extended footing based on differential burying. Cost does not include appurtenances such as ladders, hatches, and hand rails.

9.4 Treated Water Pipeline

A 20-inch diameter gravity flow treated water pipeline will supply treated water from the WHWTP to the existing distribution system. The velocity in the treated water pipeline is anticipated to be 4.2 feet per second (fps) under initial design conditions of 6.0 MGD. Under the future buildout conditions of 9.0 MGD, the velocity in the treated water pipeline will be 6.3 fps.

The Nash Road bridge overcrossing is located within the treated water pipeline alignment. The crossing is approximately 285-feet long and will provide the means by which the project's treated water line connects to the existing City distribution system. The bridge is about three years old and in nearly new condition. When the bridge was constructed, allowances were made in the bridge box (underside) to accommodate future utilities. The 20" treated water pipeline can be fit through either Bay #1 or Bay #4 in the box. Concrete coring and the use of flexible joints will be necessary for the bridge to accommodate the pipeline. Allowing the pipeline to cross under the bridge, as opposed to hanging it from the bridge side, maintains the pleasing aesthetics of the current bridge construction. Crossing the river by way of the bridge results in substantial cost savings as compared to microtunneling under the river bed. All assumptions regarding the availability of the bridge for the treated water pipeline will be verified during final design.

10 SOLIDS/BACKWASH HANDLING

This section defines the solids handling process for the waste streams generated in the treatment process. The purpose of the solids handling system is to remove water from the solids, which reduces the weight and associated hauling and disposal costs of the solids and recovers the water for treatment.

10.1 Sludge Production

Selection and design of the solids handling process is predicated on the concentration and volume of sludge produced. The two sources of solids that will require handling and disposal come from the filter backwash procedure and the Actiflo® Carb process. The amount of solids in the filter backwash will be fairly consistent since the filters will be cleaned based on elapsed time or headloss. Sludge production from the Actiflo® Carb process is dependant on the source water turbidity and organic material content. Increased turbidity and/or TOC will result in larger coagulant and PAC dosages, which lead to increased sludge production. The available historical water quality information together with the chemical doses established during the 2011 Actiflo® Carb pilot test provided the basis for this analysis. Table 10-1 summarizes the anticipated sludge production.

Table 10-1. Sludge Production

Parameter	Average (Initial)	Average (Buildout)
Flow, MGD	3	4.5
Turbidity, NTU	3.1	3.1
Polymer, mg/L	0.9	0.9
PAC, mg/L[(a)]	20	25
Ferric Chloride Dose, mg/L[(b)]	22	22
Dry Sludge, lb/day	965	1634

Notes: (a)The Average Future (Buildout) dose of PAC increases from 20 mg/L to 25 mg/L based on the assumption that future source water allocation for the WHWTP includes 50% supply from San Justo Reservoir.
(b) The average total ferric dose includes 1 mg/L post Actiflo Carb dose and BWW dose.

10.2 Solids Handling

10.2.1 Washwater Reclamation Basins

The filter backwash waste (BWW) stream is a sizeable though infrequent and short-term flow of approximately 7,200 gpm. The BWW stream flows by gravity to one of two gunite-lined earthen washwater reclamation (WWR) basins. The WWR basins are situated so that the high water level of the WWR basins is below the hydraulic gradient of the filters. This is to allow backwash water to flow by gravity to the basins. The total volume of the WWR basins is sized based on backwashing each filter once per day. Additional volume is provided to store settled solids until they are removed for drying or disposal. Water from the WWR basins will be decanted from the surface and pumped back to the head of the plant by the reclaimed eater pump station. Solids will accumulate over several months, settling to the bottom of the basins where they will thicken. The concentration achieved depends upon the depth of the solids and how long they are allowed to dry before being removed. Concentrations of 5 to 15 percent are achievable.

Settled solids will be transferred periodically with the assistance of a front-loader or flushed out a drain to the drying beds or dewatering system. Flushing the solids to drying beds or a dewatering system may dilute the solids, but subsequent drying or dewatering will achieve a drier product.

The constant waste stream from the Actiflo® Carb process will contain the majority of the solids produced by the plant. It will be less than 5 gpm with a typical solids content of 4 to 5 percent. Thus, this waste stream is suitable for discharge directly to drying beds or a dewatering system. Since prolonged wet weather could hinder drying in beds and mechanical breakdowns could suspend dewatering, provisions will be made to also discharge this stream to the WWR basins for storage.

10.2.2 Reclaim Pumps

Initially two reclaim water pumps (1 duty, 1 standby) will transfer decanted water from the WWR basins to the plant influent upstream of the Actiflo® Carb process. Space will be provided for a third pump to be added at future buildout. Either horizontal,centrifugal, end-suction pumps or submersible pumps will be used. Submersible pumps could be mounted outside, adjacent to the WWR basins where end-suction pumps would need to be placed at a lower elevation near the drying beds in order to stay primed.

Table 10-2 summarizes the preliminary design parameters for the WWR basins and reclaim pump station.

Table 10-2. WWR Basin Design Criteria

Design Parameter	6 MGD	9 MGD
Backwash Volume	320,000 gal	426,000 gal
Number of WWR Basins	2	2
WWR basin decant volume[(a)], each	216,000 gal	216,000 gal
WWR basin total volume[(b)], each	254,000 gal	254,000 gal
Length: top of basin; at water surface	120 ft; 112 ft	120 ft; 112 ft
Width: top of basin; at water surface	64 ft; 56 ft	64 ft; 56 ft
Side slope	2:1	2:1
Water depth	11 ft	11 ft
Overall depth incl. 2 ft. freeboard	13 ft	13 ft

Design Parameter	6 MGD	9 MGD
Backwash Reclaim Pumps	2	3
Capacity	300 gpm @ 20 ft TDH	300 gpm @ 20 ft TDH
Motor	3 hp	3 hp

Notes: (a) Decant volume includes the working volume that designed to hold the BWW volume from two filter cell backwashes.
(b) Total water volume is the sum of the decant volume and the sludge storage volume.

10.3 Dewatering Alternatives Evaluation

Thickened sludge from the WWR basin will require further drying prior to ultimate disposal. Drying beds and centrifuges were considered as potential options for solids thickening and drying.

10.3.1 Drying Beds

The drying beds receive a constant stream from the Actiflo® Carb process and periodically solids directly from the WWR basins. After the drying beds are filled, the solids will be allowed to dry for a period of 2 to 6 months, depending on climactic conditions. Once dry, the solids are removed from the beds with a front-end loader and hauled to the local solid waste facility.

The primary factor influencing the design loading rate for the drying beds is the climate. Based on design literature and HDR experience, wet climates are limited to a loading rate of 8 pounds per square foot (lb/ft^2) while dryer climates can typically support loading rates of up to 16 lb/ft^2. As the solids further settle in the drying beds, excess water is decanted. A center under drain provided in each lagoon aids in removal of water from beneath the sludge layer. Both decant water and under drain flow will be returned to the WWR basin through the decant pump station. The anticipated dried solids concentration at removal is approximately 25 - 45%.

An underdrain can be used to assist the performance of the drying beds. An underdrain in each drying bed aids in removal of water from beneath the sludge layer. Underdrains increase the rate at which water is removed, which would shorten each drying cycle by a week or more. However, they also increase the capital and O&M cost of the drying beds. Since there is redundancy in the amount of drying bed area the added cost and labor associated with constructing and maintain underdrains is not recommended.

The drying beds are shallow earthen basins with a soil-cement lining. The lining is needed to support a front-end loader used for solids removal. The side slopes are constructed at a slope of 2.5 to 1. A 12-foot wide roadway between each bed will be provided for mobility and a ramp into each bed will be provided for access. Figure 10-1 shows an example of a soil-cement drying bed.

Figure 10-1. Examples of Soil-Cement Lined Drying Beds

The primary advantages and disadvantages of drying beds are listed as follows:

Advantages:

- One of the oldest proven processes used to process water treatment residuals
- It is low-tech and simple to operate
- Produces up to and possibly greater than 45 percent solids sludge
- Lower relative operations and maintenance costs than mechanical dewatering

Disadvantages:

- Requires a relatively large footprint
- Performance is hindered by wet weather

- Periodic turning of the sludge may be required to improve drying; adding to the O&M costs

Table 10-3 summarizes the preliminary design criteria for the drying beds.

Table 10-3. Drying Bed Design Criteria

Item	Design Criteria
Solids Quantity (dry basis) at Average (Initial) flow of 3 MGD	965 lbs/day (initial)
Inlet Solids Concentration (from WWR Basins)	2 - 4%
Dry Solids Concentration	25-45%
Average Dry Time	4 months
Drying Lagoons Design Criteria	
Number of Drying Beds	3 (initial); 4 (future)
Surface Area (each bed)	10,000 ft^2
Surface Area (total)	30,000 ft^2 (initial); 40,000 ft^2 (future)
Depth (inches)	48
Average Loading Rate	11.8 lbs/ ft^2/yr (initial); 15 lbs/ ft^2/yr (future)

10.3.2 Centrifuge Dewatering

An alternative to drying beds is to use mechanical dewatering equipment, such as a centrifuge. Dewatering centrifuges are horizontal machines that rotate at relatively high speeds (1,000 and 3,500 rpm). A low percent solids stream is pumped at a constant feed rate into the rotating bowl where solids are separated from the liquid by centrifugal force. The liquid discharged is known as centrate. This fluid, which contains fine, low-density solids, is returned to the treatment process. The solids from the centrifuge form a cake that is discharged from the bowl by an auger. Upon discharge, the cake is conveyed to a dumpster or hopper to be hauled away for disposal. The concentration of solids dewatered with a centrifuge is usually 25 to 30 percent. Thickened solids are held for a brief period of time until enough has accumulated for efficient disposal. A dump truck or dumpster is then used for hauling to the local solids waste facility.

Centrifuges perform best with a constant feed that is consistent in concentration. Thus, a mixed feed tank is recommended upstream of the centrifuge. A feed tank improves the performance of the centrifuge and minimizes operator attention. Other associated equipment includes a screw conveyor, hopper or storage bin, and polymer feed system. The centrifuge system should be enclosed in a building or sheltered under a metal canopy to protect the equipment from the elements and improve its useful life.

The primary advantages and disadvantages of the dewatering centrifuge are listed as follows:

Advantages:

- Produces a high solids cake compared to other mechanical dewatering methods
- Combines clarification and compaction in the dewatering process
- Relatively small footprint
- May be trailer-mounted for use at multiple locations

Disadvantages:

- Higher capital and operating costs than drying beds
- Requires a mixed feed tank
- Requires a polymer system with associated chemical costs
- Significant power consumption
- Cannot achieve as dry a cake as a drying bed
- Should be enclosed or under a cover

Relative to drying beds, centrifuges require a smaller footprint. However, the O&M costs for electrical energy and daily staff attention are significantly increased. Additionally, polymer is required for efficient separation of solids and water. Capital costs of a centrifuge system are increased due to the recommended enclosure or cover.

10.4 Conclusion and Recommendations

Based on the discussions above, drying beds are recommended downstream of the WWR basins for solids dewatering. The primary advantages of this option are the lower capital and operating costs and the simplicity of operation. Space is available on the site and the drying beds can be situated away from primary view upon entering the site or hidden from view with new landscaping.

O&M costs include periodic solids removal from the beds. The John Smith landfill is eight miles from the WTP site. Hauling costs for the drying beds are anticipated to be lower than for a centrifuge, because of the higher solids concentration achievable in the summer months.

11 PLANT SUPPORT SYSTEMS

This Section discusses the architectural, landscaping, security, electrical and instrumentation systems, and site improvements proposed for the WHWTP. Solar and sustainability options are included for consideration.

11.1 Architectural

The administrative building shall be the focal point of business conducted on site accommodating the operators and maintenance staff. The building will house a control room, lab, multipurpose room, and workshop as shown in Appendix B (Figure B-11). The multipurpose room seats 8 to 10 and has a counter for coffee and complimented with a kitchen sink with garbage disposal, refrigerator, microwave oven, and dishwasher. As shown, the laboratory provides a layout typical for water treatment plants and includes a room for sample storage. The two toilet/shower rooms include lockers and are located toward the back of the building and off of an auxiliary access point. If desired, a mud room and wet storage room may be added as warranted.

The administration building is assumed to be constructed of non-bearing light steel frame construction. The supporting structure shall be a steel frame.

The exterior elevations shown in Appendix B (Figure B-12) have an uncluttered ranch house residential style in keeping with the nearby neighborhood context. This appearance not only is outwardly pleasing, but also provides opportunity for economy in the structure by allowing for pre-engineered trusses.

The roof can either consist of concrete mission tile or lightweight metal shingles. Flat roof areas will have single membrane roof. A mansard at the perimeter of the outward part of the flat roof will be ideal for hiding the view of roof top equipment.

The exterior wall material is Portland cement plaster. Windows can be storefront with larger windows divided by tube steel mullions. A tube steel header will cap off the openings. Exposed tube steel will be painted the same color as the window frames. The window on the right side at the restroom will have metal panel installed in the lower panes instead of glazing. This preserves a consistent sill and head height over all of the openings, but maintains privacy. The overhead aluminum door at the shop provides large equipment accessibility.

11.1.1 Solar and Sustainability

The WHWTP has the option to incorporate solar panels and / or other sustainable design elements into the facility design, as discussed in the following subsections.

Solar Panels

Solar panels are an option to provide a portion of the electricity needed to meet the demands of site equipment. The solar electricity will be used entirely on-site. The new facility, by design, would operate in a low power state given that the site is primarily operated by gravity. This element is very important, as adding solar power generation to an already efficient treatment plant will move the MOU Parties closer to an energy neutral operation.

Solar power can be procured in two ways. Solar panels can be purchased outright by the one or more of the MOU Parties and operated of by their staff. Alternately, a Power Purchase Agreement (PPA) can be developed. A PPA with a third-party provider is preferable to direct purchase and ownership of the solar photovoltaic (PV) system because it does not require a large capital outlay. A PPA does, however, give the MOU Parties the option of ownership through buyout at the end of the contract period. A PPA further mitigates ownership risk to the MOU Parties by eliminating performance degradation risks, design and construction contingencies, and ongoing maintenance.

Several PPA firms that service Northern California customers are listed below. The listing is not comprehensive, but provides a starting point for obtaining bids and quotes.

- Recurrent Energy
- Solar Power Partners
- SolarCity
- SunEdison
- SunPower Corporation
- Tioga Energy

The scope of the solar panel integration will be determined during the final design phase. Many options are available for panel location, including covering the administration building or adding ground-mounted systems to the surrounding hillsides. Because the PPA alternative would not impact the capital cost of the project, use of solar panels has not been added to the preliminary project cost estimate.

Sustainability

Motivations for pursuing sustainable design elements in a building project may include improving operational efficiency to help save resources, day-to-day expenses, and overall health and happiness of employees and/or garnering public support for a particular program. A valuable first step to establishing a green path that is suited to the building project is defining the project specific driving motivation(s).

In many communities, progressive sustainable policy is popular; in others it is not as important. U.S. Green Building Council's LEED program is a well known and comprehensive approach to sustainable design that has much prestige associated with it and with buildings that are awarded LEED certification. In communities where LEED certification is not essential, "LEED commissionable" may be a better approach. "Commissionable" design follows LEED standards, but LEED commission or registration is not sought. This approach saves the cost of commissioning, gains the real benefits, but does not earn a plaque in the lobby of the building saying so.

Another component of sustainable design is establishing a benchmark for each sustainable category. Depending on the approach selected, a "normal" value is established as a base to improve upon in the new design. For instance, a building that uses a certain number of kilowatts by conventional design, a goal of 10% performance improvement could be established. Metal siding that would conventionally be made of entirely new materials could now be required to contain 15% post consumer content and 10% pre-consumer content. These are just examples that help illustrate the use of a benchmark. LEED is the most recognized source of these benchmarks.

Design elements that can be considered for sustainable design include:

- Solar Panels
- Collect rain water and gray water in cisterns for use in irrigating landscaping - water savings
- Waterless urinals - water savings.
- Two flush toilets - water savings.
- Light colored roof - saves energy by reflecting solar heat gain and helps to prevent heat island effect.
- Vegetative roof - modular systems are available. Vegetated help with heat island effect and with the diversion of gray water.
- Recycled content in several materials including metal siding and interior finishes.
- Solar reflecting glass.
- Sun shades
- Promote employees to use low emission vehicles by providing prime parking spots for these vehicles.
- Showers and bike racks for employee use.
- Native plants for landscaping.
- Use of low emitting materials.
- Use of certified wood
- Use of rapidly renewable resources
- Energy efficient HVAC system

The conceptual costs included in this PDR do not currently assume the above green design elements. Based on future discussions with the MOU Parties regarding green design objectives and associated costs and benefits, these elements may be added during the design phase.

11.2 Landscaping

Landscaping within the facility will be kept to the structure and/or site perimeters to screen the facility, reduce water use, and minimize maintenance. Minimal landscaping will be provided near signage at the site entrance gate. Native varieties of trees and shrubs can be used to obscure the facility from view. No turf will be used to minimize maintenance and reduce water use. No landscaping is planned along the driveway from Union Road. Drip irrigation will be used to conserve water.

11.3 Site Security

Security measures to be included in the design of the WHWTP and RWPS focus on maintaining the facility's mission of providing safe and reliable drinking water to their customers. Electronic and physical measures are part of a security strategy to help ensure this mission is not compromised or otherwise interrupted.

The design basis threat identified for the WHWTP are low level hackers, active vandals, and domestic terrorists armed and intent upon contaminating the water supply, damaging assets, and eroding public confidence. The electronic and physical measures recommended are consistent with existing measures at or planned for other similar facilities and likewise will reduce the risk to the design basis threat.

The recommended measures are designed to deter, detect, delay, and document undesired events, which include the damage or destruction of critical components and assets. The recommendations include:

- Perimeter Fences and Gates
- Closed Circuit Video Cameras
- Intrusion Detection Alarms
- Electronic Access Control Systems
- Landscaping
- System Communication
- Chemical Storage Security
- Lighting

Perimeter fences and control gates with trespass warning signs and site lighting are designed to deter intruders; microwave devices are designed to detect unauthorized intrusion to the facilities; building walls with appropriate locks and access control devices provide delay; and video cameras supported with sufficient lighting and digital video recorders provide assessment and documentation of events.

11.3.1 WTP Perimeter

The integrity of a facility's perimeter is commonly maintained by the use of fences or walls to prevent unauthorized access to a facility and the critical assets contained within. Without this physical barrier, the territoriality of a facility and the limits of the property cannot be clearly established.

The architecture of the WHWTP and RWPS structures are being designed with as much emphasis on providing a facility that is functional as well as aesthetically pleasing. Since the fence lines of the WHWTP will be visible to the public, the fence lines will either enhance or detract from the aesthetics of the facility. Likewise, the fence line of the RWPS will be in direct view since it is adjacent to Union Road. Therefore, it is recommended that these fence lines consist of either a decorative fence or a poly coated chain-link fence. Fencing made by the Omega Corporation or a similar wrought iron style fence would provide the desired physical barrier while enhancing the appearance of the facilities. The Omega style fence is recommended because it is manufactured with a climb and cut resistant fabric while providing a non-institutional look. Figure 11-1 is a photograph of the Omega fence.

Figure 11-1. Omega Fence

Alternatively, 1-inch chain-link fencing to 8 feet in height with a 12-inch high, vertically-mounted, 3-strand barbed-wire top rigger is recommended. The fence should be installed with a bottom rail, but without top rail. By removing the top rail, climbing over the fence is much more difficult as there is nothing to grab or step onto to transition over the top of the fence. Utilizing a bottom rail provides a means to attach the fence material and prevents the bottom of the fence from being pulled up to crawl under. A 12-inch wide by 4-inch deep concrete maintenance strip is also recommended along the fence line to inhibit tunneling. Figure 11-2 shows a typical chain-link fence detail.

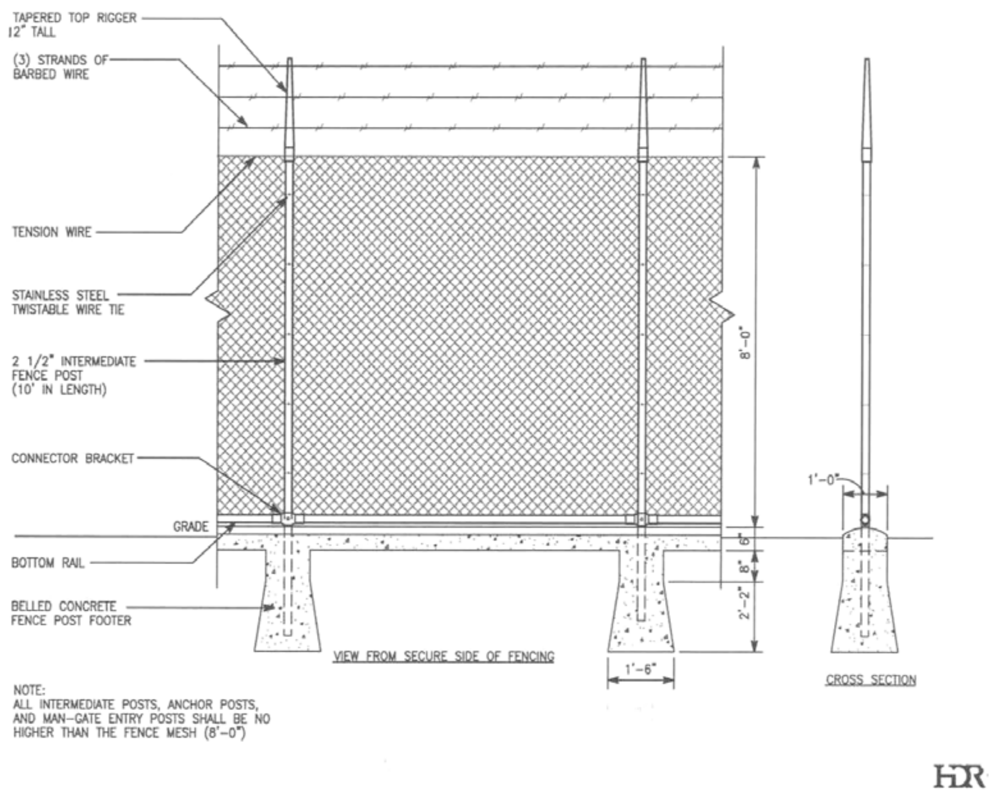

Figure 11-2. Chain-Link Fence

The recommended fences and estimated costs for the facility perimeter fence are summarized in Table 11-1.

Table 11-1. Perimeter Fence Recommendations

Security Device	Model/Type	Estimated Cost Installed
Iron Fence	Omega Fence	$37/foot
Chain-link fence with barbed wire and footer	Galvanized	$25/foot

The WHWTP will have a motorized vehicular gate at the entrance. Facility personnel would verify all non-employee business prior to admittance, control gate operations, and monitor facility security systems. Facility staff should verify and escort deliveries and guests until their departure. During elevated Homeland Security Alert Levels, more restrictive access policies could be enforced to reduce non-employee access to the WHWTP.

The main entrance will have ingress and egress stanchions equipped with proximity card readers, intercoms, and video cameras for employee access. The card reader should provide a read range of approximately 30 inches to allow the vehicles of different heights to utilize a single reader. The readers at the facility will be compatible with existing card readers installed throughout the water system. Figure 11-3 is an illustration of a typical proximity card reader, intercom, and video camera for employee, visitor, and vendor access.

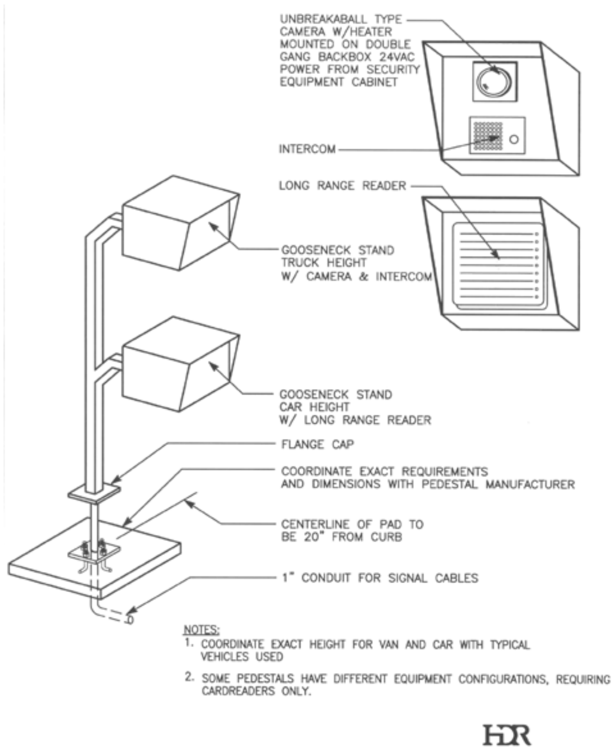

Figure 11-3. Employee Access Stanchion

The recommended employee access stanchion is summarized in Table 11-2.

Table 11-2. Employee Access Stanchion

Security Device	Manufacturer/Model #	Approximate Cost Installed
Ingress/Egress Stanchion w/Card Reader, Camera and Intercom	Paragon Metal Products/HID Maxiprox/Elbex Unbreakaball/Zenitel Intercom	$5,000

11.3.2 Intrusion Detection

Immediately contiguous to the inside of the fence line, an area free of obstructions called a clear zone must be maintained. This will be provided with native ground that extends around the full inner perimeter. This clear zone provides an area of surveillance of the entire facility perimeter. It is recommended that the clear zone be monitored with a microwave intrusion detection system utilizing a combination of bi-static and mono-static sensors. Microwave detectors mounted on 4-foot square aluminum stock poles can establish detection zones up to 500 feet. Figure 11-4 shows a typical microwave intrusion detention system.

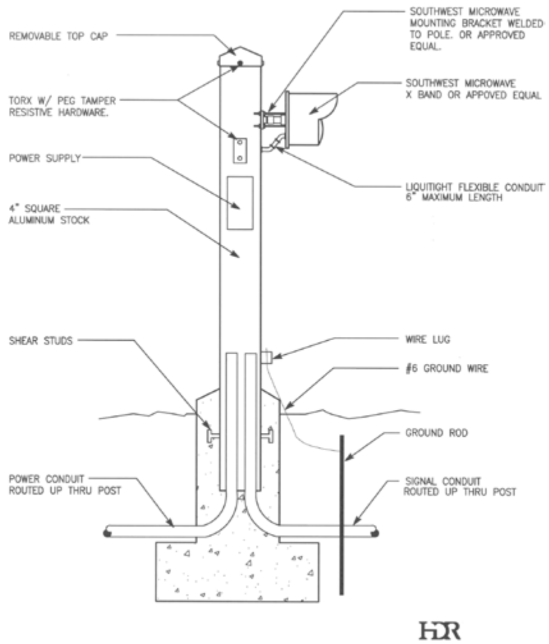

Figure 11-4. Microwave Intrusion Detection

It is further recommended that pole or parapet mounted pan, tilt, and zoom (PTZ) equipped color video cameras are provided as needed to automatically assess any intrusions of the clear zone reported by the microwave system. Lighting for the cameras could be incorporated on the mounting brackets so that they pan and tilt with the cameras. This ensures that the required lighting level will be maintained no matter where the camera is directed. During low light level situations, the camera light source could be white light, or infrared light, or both depending on the security strategy being employed. Infrared illuminators are recommended for non-obtrusive light and surreptitious surveillance.

The recommended illuminators cast sufficient non-visible illumination for camera zones up to 500 feet, which will coincide with the microwave zones. Figure 11-5 shows the type of PTZ camera proposed.

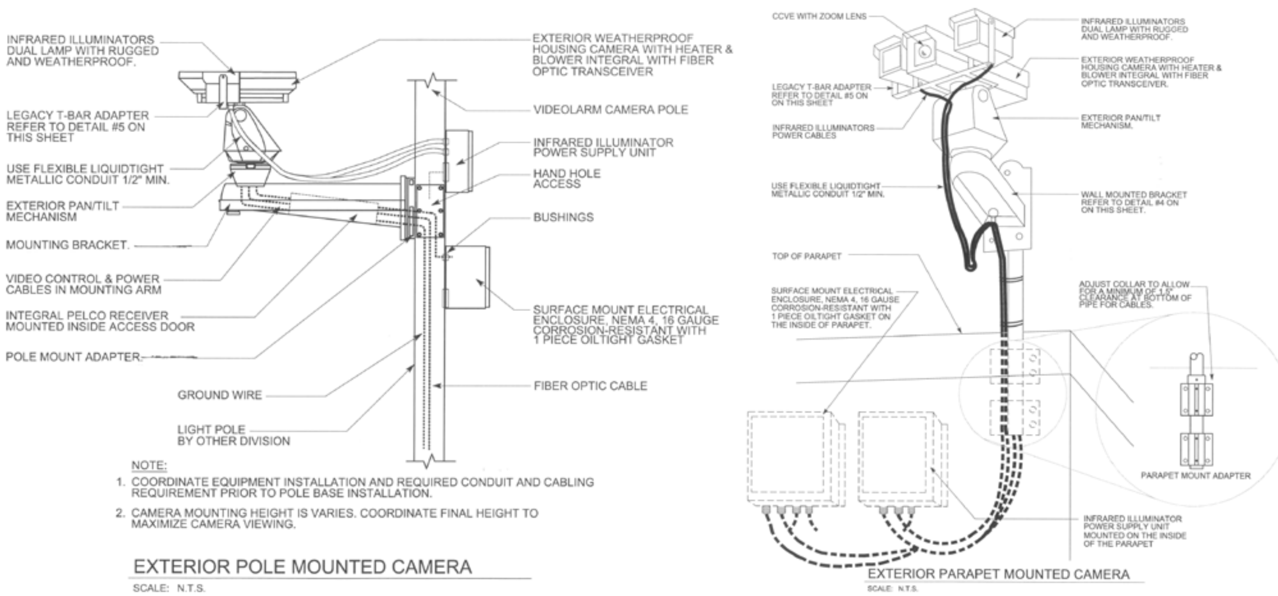

Figure 11-5. Pan Tilt, Zoom Camera

In addition to the lighting provided for video requirements, employee and visitor parking areas should be provided with security lighting at levels consistent with those recommended by the Illuminating Engineering Society of North America. Metal halide lighting is preferred to provide full color spectrum light to support color camera operations. At a minimum, the interior roads and building walls should be lit for security and video surveillance. Table 11-3 summarizes the intrusion detection equipment recommended.

Table 11-3. Intrusion Detection Equipment

Security Device	Model/Type	Estimated Cost Installed
Microwave Motion Detector/Zone	Southwest Microwave # 300B/375C	$15,000/zone
Infrared Illuminator	Extreme CCTV #UF500	$4,000/pair
Color Video Camera w/PTZ	Pelco Esprit	$4,600

11.3.3 Landscaping

Landscaping planted around the buildings should be the low-lying type to prevent hiding areas. Trees, if any, around the WTP perimeter should be limbed up 10 feet from the ground to facilitate surveillance, and should be located away from fence lines and building walls to prevent the trees from becoming an aid to climbing and/or an obstruction to video surveillance fields of view.

11.3.4 Building Access

Triple-biased door position switches will monitor all entrances to the building. Proximity card readers will be provided on specific doors based on the door's function and frequency of use. All offices and interior spaces that are designed with windows will be equipped with glass break detection. The intrusion detection, access control, and video surveillance systems can be monitored 24/7 if desired. Visitor access to the administrative offices will be restricted to the main lobby by video surveillance and access controlled doors by card readers. The design intent should be to preclude visitor access to the employee circulation of the facility. After-hours employees will utilize a proximity card reader at a designated entrance to access the facility. Request-to-exit motion sensors will permit authorized egress of the controlled doors without initiating the intrusion detection system. The recommendations for building access equipment are summarized in Table 11-4.

Table 11-4. Building Access Equipment

Security Device	Manufacturer/Model	Estimated Cost Installed
Triple Biased Door Position Switch	Sentrol #2700	$1,000
Dual Technology Motion Detector (Glass Break)	Sentrol #2T70/2T360	$1,000
Card Reader w/Electronic Lock	HID Proxpro	$4,500
Request-to-Exit Motion Sensor	Sentrol #RTE 1000	$1,000

11.3.5 Interior Sensitive Areas

Designated areas within the facility, such as server rooms, SCADA and security control centers and the chemical storage area will be contained to limit access to these security sensitive locations. Access control for these sensitive areas will be controlled by multiple technologies utilizing proximity cards and biometrics. The chemical storage area will also be provided with a closed circuit video camera triggered by a motion detector to monitor and record the activity in this area. A duress alarm will be provided for employee safety. Intrusion switches should be provided on the finished water storage tank access hatches, which should also be locked. The equipment recommendations for interior sensitive areas are summarized in Table 11-5.

Table 11-5. Interior Sensitive Areas Equipment

Security Device	Manufacturer/Model #	Estimated Cost Installed
Proximity Card/Fingerprint Reader	Bioscrypt VPROX Reader	$1,100
Duress Alarm	Alarm Control Corp #KR44L4	$500
Fixed Color Video Camera	Pelco Esprit	$1,000

11.3.6 Security System

All security related systems in the facility will be integrated into a single security monitoring system. Video surveillance, building perimeter and microwave intrusion sensors, an intercom at the entrance and access control systems will interface with the system to provide seamless monitoring and operation of all devices. Access control workstations will be provided at the security control center in the administrative building to locally monitor the security systems. The major components of the security system are summarized in Table 11-6.

Table 11-6. Security System Integration Components

Security Device	Manufacturer/Model #	Estimated Cost Installed
Digital Video Recorder	Pelco DX 7000	$3,500
Video Switcher/Controller	Pelco CM 6800	$6,000
18" Color LCD Flat Panel Monitor	Philips 180P2G	$800
Intercom System	Zenitel	$4,000
Access Control Workstation	AMAG Professional Edition	$5,000
Access Control Remote Panel	AMAG #M2100	$2,500

The security measures recommended herein will provide the necessary physical and electronic devices to defend against the identified design basis threat. However, employee security awareness training, coupled with well-defined policies and procedures, are as important to the security of a facility as the recommended devices. A holistic approach to security is proposed and security policies and procedures shall be specifically designed for the operation of the new facility. The policies and procedures should be developed by security and operations staff prior to the facility's commissioning in order to provide guidance to employees on the first day of operation. Every employee should then be provided a copy or computer access to the policies and procedures and be required to follow them, under penalty of disciplinary action, in order to insure compliance and maintain the integrity of security at the facility.

11.4 Plant Electrical Instrumentation Systems

11.4.1 Power Requirements

The electrical service and distribution system shall be designed to handle the initial electrical load and the future plans to expand the plant to 9 MGD. The WHWTP consists of two electrical services, the treatment plant and the raw water pump station located near the plant entrance on Union Road.

Water Treatment Plant

The main utility service to the WHWTP shall be extended from Union Road up the entrance driveway overhead to a service pole located at the WTP. The service will drop underground at that point to a pad-mounted transformer located near the electrical room. The pad-mounted transformer shall be rated for 500 KVA. The WHWTP shall require an 800 ampere, 277/480 volt electrical service. See Table 11-7 for the operations loads. The total expected demand is 230 kVA.

Table 11-7. Electrical Load Estimate for Treatment Plant

Equipment	Connected (HP)	Demand	Notes
Actiflo	32	32	Duty
Actiflo	32	32	Duty
Backwash Pump	75	75	Duty
Backwash Pump	75		Standby
Backwash Pump	75		Future
Miscellaneous Pumps	7.5	7.5	Duty
Miscellaneous Pumps	7.5	7.5	Duty
Miscellaneous Pumps	7.5	7.5	Duty
Miscellaneous Pumps	7.5	7.5	Duty
Miscellaneous Pumps	7.5		Standby
Miscellaneous Pumps	7.5		Standby
Miscellaneous Pumps	7.5		Standby
Miscellaneous Pumps	7.5		Standby
Auxiliary Loads	10	10	HVAC
	10	5	Lighting, Instrumentation and Control Equipment
25%	92	46	
Total Power KVA	461	230	
Total Current Amps	554	277	

Provisions shall be provided to install a stand-by power connection for a portable generator to be connected to the main switchboard. A manual transfer switch shall provide to isolate the electric utility when the generator is powering the plant system bus.

Raw Water Pump Station

The RWPS is located at a newly acquired site at the base of the entrance driveway at Union Road. The RWPS shall require its own electrical service. The service shall be provided from a pole located near Union Road. The service shall drop underground from the service pole and continue to a pad-mounted transformer rated for 300KVA. The RWPS shall require a 400 ampere, 277/480 volt electrical service. The RWPS electrical distribution system shall be located in a single NEMA 3R enclosure that shall house the service meter, main distribution block, breakers, VFD’s, PLC, manual transfer switch (MTS), and lighting panel. The distribution block shall be rated for 600 Amp and provide power to the RWPS loads. The RWPS main loads consist of four 75 Hp vertical turbine pumps. The initial project shall consist of three pumps and provide room for one future 75 Hp vertical turbine pump. See Table 11-8 for the summary of the estimated pump station loads.

The NEMA 3R enclosure shall be covered by a canopy to help cool the equipment. The VFD’s shall have fans for additional cooling of the VFD’s.

The RWPS shall have a manual transfer switch (MTS) connected to the distribution block. A tap shall be provided for a portable generator outside the NEMA 3R enclosure. The chemicals fed into the raw water line will be housed in a small building next to the RWPS. This building shall have small loads such as security cameras, lights, and receptacles.

Table 11-8. Electrical Load Study for Raw Pump Station

Equipment	Connected (HP)	Demand (Hp)	Notes
RW Pump	75	75	Duty
RW Pump	75	75	Duty
RW Pump	75		Standby
RW Pump	75		Future
208/120 Panel	15	8	Lights, Receptacles, and Security
25%	104	52	
Total Power KVA	419	210	
Total Current Amps	504	252	

11.4.2 Motors

All motors shall be NEMA premium efficiency type. Where applicable motors shall be driven by VFD's and shall be VFD rated.

11.4.3 Control System

All equipment shall be controlled in AUTO from a central SCADA system to include vendor PLC's connected back to a central control room in or near the electrical room. Communication provisions shall be installed along the plant pipeline to allow the RWPS to be controlled from the plant as well.

11.4.4 Electrical and Control Rooms

The WHWTP electrical room shall house the MCC, soft starters, VFD's and lighting panels. A control room shall be located next to the electrical room and shall contain the SCADA workstation and possibly the Main PLC panel. The electrical and control room shall be environmentally controlled to maintain a temperature suitable for electronic equipment. The Lessalt WTP shall be connected to the WHWTP with the use of radio or leased line communications.

11.5 Civil Site Improvements

11.5.1 Site Survey

The existing survey (including two foot contours) of the WHWTP site was developed initially by Mackay and Somps, CDM, and Aerometric Surveys in 1999 as part of the Storm Drain

Master Plan 2001. For the preparation of this PDR, San Benito Engineering and Surveying, Inc. performed a survey of the site to verify the existing coordinate system, locate proposed structures on the map, and develop a base map of the site with property lines, easements, utilities, and surrounding roads. Additional survey information may be required for final design.

11.5.2 Earthwork

Geotechnical engineering work is scheduled at the site for October, 2011. Following completion of the site work, Geo Engineers, Inc. will complete a geotechnical engineering report for use in the development of plans and specifications for the WHWTP design. The report will include geotechnical recommendations for site preparation and grading, foundations, retaining walls, slabs-on-grade and exterior flatwork, utility trenches, site drainage, and finish improvements. The results of this study and the recommendations within will be incorporated in the earthwork requirements for the project.

Flood Insurance Rate Maps (FIRM) maps indicate that both the WTP and RWPS sites are beyond the 100-yr. flood elevation.

Excavation is anticipated for the treated water storage tank, administration building, filters, lagoons, and pipelines. The amount of excavation will be determined based on the results of the geotechnical report. It is anticipated that some of the excavated material will be suitable for use as fill elsewhere on the site. Due to the presence of organic material, this soil will need to be tested and meet the criteria defined in the geotechnical report prior to use.

11.5.3 Access and Parking

The entrance to the property will be from Union Road approximately one-half mile northwest of San Justo Road. The volume of traffic visiting the WHWTP is expected to be minor. Most visitors are expected to arrive by automobile; however, a few large trucks will arrive for deliveries, maintenance, and construction.

There are 2 options for the entrance road alignment. One is an existing 60-ft wide easement from Union Road to the site. The other is an existing dirt driveway immediately adjacent to the easement, called Richardson Rd. Richardson Rd. is approximately 12 ft wide and used by the landowner south of the treatment plant site. It is recommended that the MOU Parties attempt to coordinate with the landowner to widen and improve Richardson Rd. for access to the site. This would be least costly means to construct an access. As an alternative, a new road could be constructed completely within the 60 ft easement. This alternative is anticipated to require greater excavation and grading, which would increase the construction cost.

Based on the existing survey information, Richardson Rd. appears to overlap the existing 100 ft set-back from the designated vernal pool on the site, as previously defined in the existing CEQA document. Any improvements to this road or construction of a new road in the easement apparently affect this designated area. Thus, prior to finalizing the design of the access road, coordination with the ongoing revised CEQA work is essential. If not permissible based on the revised CEQA, the site entrance must be to the west of the vernal pool boundary, which would require significant grading cuts and retaining walls.

Eight standard mixed use parking stalls and one space reserved for disabled persons will be located at the administration building for employees and visitors. The driveway will loop past the administration building, filter area, and solids lagoon and back to the entrance. This route will also serve as fire access and for deliveries.

11.5.4 Signage, Striping, and Lighting

Directional signs on Union Road at the driveway will guide vehicles to the plant entrance. Parking signs will be placed on the site to delineate ADA, visitor, and staff parking areas.

Striping on Union Road will be modified to permit a left turn to the driveway. The center of the driveway will be striped as will the plant site for delineation of fire lanes and parking stalls.

The exterior of the facility will be lit with pole and wall-mounted lights. No street lighting is planned for the driveway from Union Road to the plant. No signalization is anticipated at turn off from Union Rd.

11.5.5 Road Section

The driveway will have a paved width of 20 feet. The road will be crowned in the middle with a maximum 2 percent cross-slope to the shoulders, which will be graveled.

The road looping within the plant will be a minimum of 24-foot wide with a maximum cross-slope of 2 percent to the borders for drainage. The road will be bordered with a concrete edge to retain the pavement and to separate the material in the borders. The concrete edges will be cross-sloped to match the crown in the road so as to freely permit drainage to the gravel areas. Concrete curbs will be constructed along the edge as needed to direct run off.

A structural section for the access road and pavement will be determined using tests conducted during the geotechnical investigation and expected traffic type and frequency. The structural section will be designed for a 20-year service life. The anticipated traffic at the facility will be mostly automobiles and light trucks with an occasional semi truck or bus. It is assumed that a semi truck will visit the site once a week, which equates to a 20-yr ESAL (equivalent single axel load) of 1900. The structural section will use Type B AC and Class II AB. AB composed of recycled material will be use if available.

The driveway will allow adequate access for trucks to enter, maneuver, and leave the site. It is important to consider all types of trucks that will need access. Typical truck dimensions and turning radii are shown in Table 11-9.

Table 11-9. Truck Dimensions and Turning Radii

Vehicle	Description	Width	Length	Radius[a]	Turning Template[b]
California Legal Design Vehicle 50’	Truck tractor-semitrailer allowed on all state roads. 50’ radius is tightest turn that the vehicle can navigate assuming a speed of less than 10 mph.	8.5	65	50	404.5D
California Legal Design Vehicle 60’	Truck tractor-semitrailer allowed on all state roads. 60’ radius is more conservative, vehicle is not required to stop before entering intersection.	8.5	65	60	404.5E
40’ Bus Design Vehicle	Applicable to 3-axle delivery trucks, typical of a city transit bus.	8.5	40	42.4	404.5F

Notes:
a) Radius to outside wheel at beginning of curve
b) Caltrans Highway Design Manual Turning Template Figure Number. These figures are included in Appendix B.

11.5.6 Site Piping

Process Piping

The RWPS will deliver water to the WHWTP through a 20-inch pipe that will connect directly to the Actiflo® Carb basin. The Actiflo® Carb basin effluent will be piped directly to the filters from which water will flow by gravity to the clearwell. The finished water will flow to the distribution system by gravity. The finished water pipeline will exit the WHWTP to the south and connect to the existing distribution system at the corner of Nash Rd. and Line St.

Chemical Piping

All chemical piping installed outside of the chemical containment areas will be installed in double-contained piping. Tubing will be used within the contained piping to simplify removal for repairs.

Water and Sewer

Potable water service for the WTP will come from the treated water storage tank outlet. A standard 6” service connection will be used to isolate the service from the distribution system. A small package booster pump station will be used to pressurize the system. Backflow prevention devices will be installed on the potable water service for fire connections and irrigation.

A septic system with a leach field will be installed to handle sewer flows from the plant. The system will be sized for the buildout occupancy of 3 employees plus 10 visitors. The leach field will be located based on recommendations from the geotechnical engineer.

11.5.7 Sediment and Erosion Control

Stormwater collection at the facility will comply with the County of San Benito’s stormwater design requirements. Standard erosion control and storm water pollution prevention BMPs will be required during construction. Construction BMPs will conform to San Benito County’s

Stormwater Management Program and may include fiber rolls, slope tracking, silt fences, and protected equipment staging areas. The entire site will be graded with slopes towards graveled borders, which will capture runoff from the site. Borders around the perimeter will also provide areas for vegetative landscaping.

11.5.8 Fencing

The type of fencing recommended is discussed in previously under Site Security. Decorative fencing, similar to that made by the Omega Corporation or a 1" mesh chain-link fence is recommended. The entrance will have an automated rolling iron gate.

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12 CODES AND STANDARDS

All new structures designed and constructed in California are required to the meet the 2010 State of California Building Code and local ordinances. Following is a list of codes from Title 24 of the California Code of Regulations that apply to this project.

- 2010 California Building Code (CBC)
- 2010 edition of the California Electric Code
- 2010 edition of the California Mechanical Code
- 2010 edition of the California Plumbing Code
- 2010 edition of the California Fire Code
- 2010 edition of the California Energy Code
- 2010 California Green Building Standard Code

Cal-OSHA contains provisions applicable to this project. Occupant safety provisions will be governed by Cal-OSHA requirements. In addition to the requirements of Title 24 and Cal-OSHA, several industry standards and codes will be used in designing the WTP. Following is a brief summary of the applicable standards and studies anticipated for the project.

12.1 Fire Code

The National Fire Protection Association (NFPA) contains provisions applicable to this project. Hazardous materials, fire flow provisions, and emergency vehicle access will be governed by NFPA requirements.

12.2 Geotechnical Design Criteria

A geotechnical study is scheduled for October 2011 to explore and evaluate the surface and subsurface conditions at the site and to develop geotechnical information and design criteria for the proposed project. The study includes the following:

- A review of geotechnical and geologic data available at the time of the study.
- A field study consisting of a visual site reconnaissance, followed by an exploratory boring program to characterize the subsurface conditions.
- A laboratory testing program performed on representative samples collected during the field study.
- Engineering analysis of the data and information obtained from the field study, laboratory testing, and literature review.

- Development of recommendations for site preparation and grading, and geotechnical design criteria for foundations, slabs on grade, retaining structures, underground facilities, and pavements.

A copy of the geotechnical report will be included in Appendix J, when it becomes available.

12.3 Equipment and Material Standards

The mechanical system shall be designed and built to the following codes and standards:

- San Benito County standards and ordinances
- Underwriters Laboratories Inc.
- U.S. Department of Labor Occupational Safety and Health Act
- NFPA Codes and Standards
- American Iron and Steel Institute Standards
- Anti-Friction Bearing Manufacturers Association
- American Society for Testing and Materials
- American Society of Mechanical Engineers, Boiler and Pressure Vessel Code (ASME P&PV), 1995 edition
- American National Standards Institute
- American Water Works Association

12.4 Structural Design

Structural designs fall under the jurisdiction of the 2010 CBC with California Amendments as discussed above. All codes, standards, and specifications referenced in the CBC are applicable. A sample document of the structural design standards to be used on the design of the West Hills WTP is in Appendix K. A list of additional standards to be followed in design is provided below.

- Applicable San Benito County ordinances.
- *Aluminum Design Manual*, Aluminum Association.
- Building Code Requirements for Masonry Structures (ACI 530-05/ASCE 5-05/TMS 402-05).
- Building Code Requirements for Structural Concrete (ACI 318-05).
- Code Requirements for Environmental Engineering Concrete Structures by the American Concrete Institute (ACI 350-06).

- Design Considerations for Environmental Engineering Concrete Structures by the American Concrete Institute (ACI 350.4R-04).
- Foundations for Dynamic Equipment by American Concrete Institute (ACI 351.3R-04).
- Minimum Design Loads for Buildings and Other Structures. American National Standards Institute (ANSI)/American Society of Civil Engineers (ASCE) 7-05.
- Seismic Design of Liquid-Containing Concrete Structures and Commentary (ACI 350.3-06) by the American Concrete Institute.
- Steel Construction Manual, 13th edition by the American Institute of Steel Construction (AISC).
- U.S. Department of Commerce, National Technical Information Services, U.S. Atomic Energy Commission, Washington D.C., Nuclear Reactor and Earthquake, TID-7024, Chapter 6, Dynamic Pressure of Fluid Containers, August 1963.

12.5 Architectural Design

In addition to the requirements of Title 24 listed above, the following codes and standards apply to the architectural design of this subject:

- San Benito County Ordinances
- Accessible and Usable Buildings and Facilities 2003 (ICC/ANSI 117.1-2003)
- State of California Occupational Safety and Health Administration (CAL/OSHA) General Industry Safety Orders
- Federal Occupational Safety and Health Administration (OSHA)
- Americans with Disabilities Act (ADA) The ADA Accessibility Guidelines for Buildings and Facilities

Design of the facilities and the materials of construction will comply with the standards of the following entities:

- ADA Accessibility Guidelines for Buildings and Facilities
- American National Standards Institute (ANSI)
- American Society for Testing and Materials (ASTM)
- National Association of Architectural Metal Manufacturers (NAAMM)
- Sheet Metal and Air Conditioning Contractors National Association (SMACNA)
- Architectural Sheet Metal Manual 2003
- Steel Structures Painting Council

- Factory Mutual Research Corporation
- Underwriter's Laboratory, Inc.

12.6 Heating, Ventilation, and Air Conditioning

In addition to the requirements of Title 24 listed above, the following codes and standards apply to the HVAC design:

- American Society of Heating, Refrigerating, and Air-Conditioning Engineers (ASHRAE) Standards 90.1-2004. – Energy Standard for Buildings Except Low Rise Residential Buildings
- ASHRAE Standard 62.1 - 2007 – Ventilation for Acceptable Indoor Air Quality
- ASHRAE, HVAC Applications Handbook - 2007
- NFPA Standard 90A, "Installation of Air Conditioning and Ventilation Systems"
- Industrial Ventilation: Handbook of Recommended Practice – 2007
- American Industrial Hygiene Association (AIHA) ANSI/AIHA Standard Z9.5-93, Laboratory Ventilation - 2003
- California Energy Commission (CEC) Title 24, Energy Efficiency Standards, 2005
- SMACNA Duct Construction Standards – Metal and Flexible 2006
- OSHA
- NFPA Standard 820, Standard for Fire Protection in Wastewater Treatment and Collection Facilities. 2008
- SMACNA – Thermostat FRP Duct Construction Manual

12.7 Plumbing

In addition to the requirements of Title 24 listed above, the plumbing system shall be designed and built to the following codes and standards:

- International Plumbing Code, 2006 edition
- ANSI
- ASHRAE
- HUD Standards for Energy Conserving Appliances

12.8 Electrical

In addition to the requirements of Title 24 listed above, all electrical construction will be performed in the accordance with the most current version of the following codes and standards:

- San Benito County Standards and Ordinances
- NFPA and NEC
- Design to comply with requirements for NFPA 820 and NFPA 70
- ANSI
- ICEA
- Institute of Electrical and Electron Engineers (IEEE)
- Instrument Society of America (ISA)
- National Electrical Manufacturers Association (NEMA)
- Association Edison Illuminating Companies (AEIC)
- Occupational Safety and Health Administration Standards (OSHA)
- Insulated Power Cable Engineers Association (IPCEA)
- Underwriters Laboratories, Inc. (UL)
- Pacific Gas and Electric (PG&E)

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13 Opinion of Probable Project Costs

This Section Provides the basis for cost estimating, capital costs, and operation and maintenance costs.

13.1 Cost Estimating Procedures

Throughout the project, the following procedures are utilized during the cost estimation process to ensure that the estimates reflect the design and current construction climate:

- Generate opinions early in the design phase, and continuously update to reflect design changes and cost changes.
- Maintain information on the construction climate both locally and nationally.
- Use recognized cost estimating software in conjunction with national cost databases, customized to reflect the most current cost information.
- Obtain direct equipment manufacturing budget information on major engineered equipment.
- Perform joint reviews of project cost estimates by the design and estimating teams, to ensure the estimate accurately reflects the project scope and design intent.
- Perform QA/QC on cost opinions.

The opinions of probable construction and operating costs for the WHWTP presented herein were developed (as described above) from the preliminary design criteria described within this PDR, budgetary quotes from major equipment suppliers, standard cost estimating guidelines, and engineering experience with similar projects.

Table 13-1 presents a summary of cost estimating level descriptions, accuracy, and recommended contingencies based on the development level of the project. These data were compiled based on the standards of the Association for the Advancement of Cost Engineering (AACE).

Table 13-1. Cost Estimating Guidelines

Estimate Class	Class 5		Class 4		Class 3		Class 2		Class 1	
LEVEL OF PROJECT DEFINITION Expressed as a % of complete definition	0% to 2%		1% to 15%		10% to 40%		30% to 70%		70% to 100%	
END USAGE Typical Purpose of Estimate	Concept Screening or Feasibility		Concept/Alternatives Study or Feasibility		Budget Authorization, or Control		Control or Bid / Tender		Check Estimate or Bid / Tender	
METHODOLOGY Typical estimating method	Timberline Model Estimating, Parametric Models, Judgment, Equipment Budgets		Timberline Model Estimating, Parametric Models, Selective Deterministic, Equipment Budgets		Selective Deterministic, Timberline Assembly Estimating, Equipment Budgets		Primarily Deterministic with Detailed Unit Cost with Forced Detailed Take Off, Vendor Quotes		Deterministic with Detailed Unit Cost and Detailed Take-Off	
EXPECTED ACCURACY RANGE Typical variation in low and high ranges [a]	L: Range -20% to -50%; Typically -25%	H: Range +30% to +100%; Typically +50%	L: Range -15% to -30%; Typically -20%	H: Range +20% to +50%; Typically +40%	L: Range -3% to -10%; Typically -5%	H: Range +3% to +15%; Typically +10%	L: Range -5% to -15%; Typically -10%	H: Range +5% to +20%; Typically +20%	L: Range -3% to -10%; Typically -5%	H: Range +3% to +15%; Typically +10%
UNDEFINED SCOPE OF WORK ESTIMATE DEFINITION (Contingency)	Cost included in the OPCC Estimate which can not otherwise be allocated to specific task due to lack of Project Definition assuming relative stability of project scope and assumptions upon which the estimate is based.									
UNDEFINED SCOPE OF WORK ESTIMATE %	Range 25% to 40%; Typically 25%		Range 20% to 30%; Typically 20%		Range 15% to 25%; Typically 15%		Range 10% to 20%; Typically 10%		Range 5% to 15%; Typically 5%	
EXPANDED CLASS DEFINITION	Class 5 estimates are generally prepared based on very limited information, and subsequently have very wide accuracy ranges. As such, some companies and organizations have elected to determine that due to the inherent inaccuracies, such estimates cannot be classified in a conventional and systematic manner. Class 5 estimates. Often, little more than proposed site layout, plant type, location, and capacity are known at the time of estimate preparation. Special site conditions (ex. rock, piles, environmental, etc) are not taken into account at this level.		Class 4 estimates are generally prepared based on limited information, and subsequently have wide accuracy ranges. They are typically used for alternatives or concept screening, determination of feasibility, concept evaluation, and preliminary budget approval. Typically, engineering is from 1% to 15% complete, and would comprise at a minimum the following: capacity, block schematics, indicated layout of structures and piping, sized process flow diagrams (PFDs) for main process systems, preliminary motor and instrument lists, and preliminary engineered process and utility equipment lists. Estimates may be limited to compare alternatives and not indicative of project cost.		Class 3 are generally prepared based on average design development level of 30% and as such are prepared to form the basis for budget authorization, appropriation, and/or funding. They typically form the initial cost control estimate against which all actual costs and resources will be monitored. Engineering development for different areas will range from 10% to 40% complete, minimum design documents would comprise of the following: sized process flow diagrams (PFDs), P&ID's, site plan, preliminary yard piping plan, developed facility layout drawings and initially sections, complete motor and instrument list, and complete engineering process and major utility equipment lists.		Class 2 estimates are prepared based on average design development level of 60% and as such are prepared to form a detailed control baseline against which all project work is monitored in terms of cost. Engineering development for different areas will range from 30% to 75% complete, minimum design documents would comprise of the following: Process flow diagrams(PFDS), utility flow diagrams, piping and instrument flow diagrams (P&IDS), heat and material balances, final site plan, final yard piping plan, final layout drawings, significant sections, complete engineered process and utility equipment lists, single line diagrams for electrical, final electrical equipment and motor schedules, vendor quotations, detailed project execution plans, resourcing and work force plans, detailed demolition plans,etc.		Class 1 estimates are generally prepared based on average design development of 95% for the total project and as such are used to establish the final cost of the project. This estimate is often referred to as the final or current control estimate and becomes the baseline for cost/schedule control of the project. Class 1 estimates may be prepared for parts of the project to comprise a fair price estimate or bid check estimate to compare against a contractor's bid estimate, or to evaluate/dispute claims. Engineering development for different areas will range from 50% to 100% complete, and would comprise virtually all engineering and design documentation of the project, and complete project execution and commissioning plans.	
ESTIMATING METHODS USED	Class 5 estimates virtually always use stochastic estimating methods such as cost/capacity curves and factors, Timberline Model Estimates, or in house capacity curves for similar plants, prior experience with similar projects, or other parametric and modeling techniques.		Class 4 estimates are frequently a mix of forced deterministic, Timberline, and stochastic estimating methods such as cost/capacity curves and factors, Timberline Model estimating, gross unit costs/ratios. Use of budget quotes from engineered equipment vendors is recommended.		Class 3 estimates usually involve more deterministic, timberline, estimating methods that stochastic methods. They usually involve a high degree of unit cost line items, although these may be at an assembly level of detail rather than individual components. Factoring and other stochastic methods may be used to estimate less-significant areas of the project.		Class 2 estimates always involve a high degree of deterministic, Timberline, estimating methods. Class 2 estimates are prepared in great detail, and often involve tens of thousands of unit cost line items. For those areas of the project still undefined, an assumed level of detailed takeoff (forced detail) may be developed to use as line items in the estimate instead of relying on factoring methods.		Class 1 estimates involve the highest degree of deterministic, Timberline, estimating methods, and require a great amount of effort. Class 1 estimates are prepared in great detail, and thus are usually performed on only the most important or critical areas of the project. All items in the estimate are usually unit cost line items based on actual design quantities.	
EXPECTED ACCURACY RANGE	Typical accuracy ranges for Class 5 estimates are -20% to 50% on the low side, and +30% to +100% on the high side.		Typical accuracy ranges for Class 4 estimates are -15% to -30% on the low side, and +20% to +50% on the high side,		Typical accuracy ranges for Class 3 estimates are -10% to -20% on the low side, and +10% to +30% on the high side.		Typical accuracy ranges for Class 2 estimates are -5% to -15% on the low side, and +5% to +20% on the high side.		Typical accuracy ranges for Class 1 estimates are -3% to -10% on the low side, and +3% to +15% on the high side.	

13.2 Conceptual Level Capital Costs

The design for the WHWTP has been developed to a conceptual/preliminary (approximately 10%) level, with preliminary design criteria, conceptual site and building plans, and a basic understanding of site conditions and limitations. Therefore, the level of accuracy for the capital and operating cost estimate presented should be considered to represent a Class 4 estimate. An estimate contingency of 20 percent, reflecting that used with a Class 4 estimate was applied to the opinion of probable construction cost.

The cost estimate also includes markups for taxes on materials of 9.25 percent, a 7 percent markup for field general conditions (including mobilization), 1.5 percent for bonding and insurance, and a 10 percent markup for general contractor overhead and profit.

Table 13-2 provides a summary of the preliminary opinion of probable construction cost of the facilities for the WHWTP. The total probable capital cost is $18,614,000. The total project capital cost (including engineering and construction services) is $22,064,000. A detailed break down of the cost assumptions and estimate by work area is attached in Appendix L.

Table 13-2. Preliminary Opinion of Probable Capital Costs

Cost Components	Raw / Treated Water Pipelines	Raw Water Pump Station	West Hills WTP
Field General Conditions	$ 153,000	$ 70,000	$ 930,000
Sitework and Yard Piping	$ -	$ -	$ 1,113,000
Raw Water Pump Station		$ 534,000	$ -
Raw Water Pipeline	$ 958,000	$ -	$ -
Actiflo/Carb		$ -	$ 1,940,000
Gravity Filters (Concrete)	$ -	$ -	$ 1,069,000
Treated Water Storage Tank	$ -	$ -	$ 640,000
Treated Water Pipeline	$ 1,891,000	$ -	$ -
Backwash Supply Pump Station	$ -	$ -	$ 40,000
Backwash Waste EQ Basin	$ -	$ -	$ 192,000
Backwash Waste Pump Station	$ -	$ -	$ 82,000
Return Water Pump Station	$ -	$ -	$ 40,000
Solids Lagoons Pump Station	$ -	$ -	$ 40,000
Solids Lagoons	$ -	$ -	$ 512,000
Decant Pump Station	$ -	$ -	$ 40,000
Chemical Feed Facility	$ -	$ -	$ 344,000
PAC System	$ -	$ -	$ 501,000
Operations Building	$ -	$ -	$ 543,000
I&C	$ -	$ 48,000	$ 642,000

Cost Components	Raw / Treated Water Pipelines	Raw Water Pump Station	West Hills WTP
Electrical	$ -	$ 85,000	$ 1,124,000
Sales Tax (9.25%)	$ 200,000	$ 35,000	$ 390,000
Contractors Fee (10%)	$ 320,000	$ 77,000	$ 1,018,000
Bonds and Insurance (1.5%)	$ 48,000	$ 12,000	$ 153,000
Contingency (20%)	$ 640,000	$ 154,000	$ 2,036,000
Construction Cost	$ 4,210,000	$ 1,015,000	$ 13,389,000
Total Construction Cost	$ 18,614,000		
Design (8%)	$1,489,000		
Construction Management (10%)	$ 1,861,000		
Property Acquisition	$100,000		
Total Capital Cost	$ 22,064,000		

13.3 O&M Costs

Table 13-3 provides a summary of the preliminary opinion of probable annual operations and maintenance costs. The probable annual operations and maintenance cost is $1,273,000, assuming the source water supply is equally split between SLR and SJR. A detailed break down of the O&M cost assumptions is attached in Appendix L.

Table 13-3. Preliminary Opinion of Probable Annual O&M Costs

Cost Components	Raw / Treated Water Pipelines Raw Water Pump Station West Hills WTP
Power [(a)]	$ 197,000
Chemicals[(b)]	$ 614,000
Filter Media Replacement	$ 25,000
Solids Hauling & Disposal [(c)]	$ 26,000
Labor [(d)]	$ 200,000
Misc. (Maintenance, ops building, instrumentation, lab testing, etc at 20%)	$ 191,000
Total O&M Cost ($/yr)	$ 1,273,000

Notes:
a) Assumes $0.17/kWhr.
b) Assumes source water supply is from SJR 50% of year and from SLR 50% of year.
c) Assumes $25/CY disposal.
d) Assumes 2.5 operators.

13.4 Life Cycle Costs

Table 13-4 summarizes the preliminary opinion of probable project life cycle cost, based on the construction cost and present worth of the annual operating costs. The present worth of the annual operating costs was computed assuming a 3 percent discount rate and a 20 year period of analysis. The probable project life cycle cost is $42,283,000. The costs are 2011 dollars.

Table 13-4. Preliminary Opinion of Probable Project Life Cycle Cost

Cost	Raw / Treated Water Pipelines Raw Water Pump Station West Hills WTP
Capital	$ 22,064,000
Annual O&M	$ 1,273,000
Present Worth O&M	$ 18,641,000
Total Life Cycle Cost	$ 42,283,000

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14 PROJECT SCHEDULE AND PERMITTING

Establishing a realistic, agreed upon project schedule is a critical element of successful project implementation. A permitting action plan, developed at the start of final design, describes the approach, sequence, and timing required for securing all necessary project specific permits. This section presents both a proposed schedule and a list of permits for inclusion in a permit action plan.

14.1 Schedule

After review and approval of the preliminary design report, the project will move forward into the detailed design phase. The preliminary schedule anticipates the 30 percent design being completed in early February 2012, and completion of the final design in the late summer of 2012. The project schedule assumes implementation of the standard design-bid-build process. The proposed schedule is shown below in Figure 14-1.

ID	Task Name	Start	Finish
1	**Design**	**Mon 1/9/12**	**Mon 10/1/12**
2	Notice to Proceed	Mon 1/9/12	Mon 1/9/12
3	30 % Design	Tue 1/10/12	Mon 4/16/12
4	30 % Review	Mon 4/16/12	Mon 4/30/12
5	60 % Design	Tue 5/1/12	Mon 7/2/12
6	60% Review	Tue 7/3/12	Mon 7/16/12
7	90% Design	Tue 7/17/12	Mon 9/17/12
8	90% Review	Tue 9/18/12	Mon 10/1/12
9	Advertise for Bids	Tue 10/2/12	Mon 10/29/12
10	Bid Period	Tue 10/30/12	Wed 12/12/12
11	Construction	Thu 12/13/12	Thu 1/8/15
12	Final Testing and Start-up	Fri 1/9/15	Thu 4/2/15

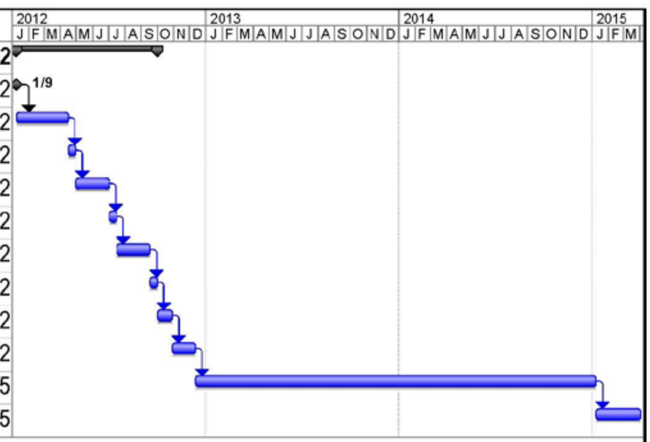

Figure 14-1. Preliminary Project Schedule

14.2 Regulatory Requirements and Permits

Table 14-1 lists the various federal, state, local, and other permits/approvals that would be required for construction and operation of WHWTP. The remainder of this Section describes each specific permit requirement.

Table 14-1. Summary of Regulatory Requirements and Permits

Agency	Type of Approval	Project Component
Federal		
US Bureau of Reclamation	Connection to Hollister Conduit	Raw water pipelines
State Agencies		
Central Coast Regional Water Quality Control Board	National Pollutant Discharge Elimination System (NPDES) Construction Storm Water Permit	WTP, RWPS, and raw and treated water pipelines
	General Order for Dewatering and Other Low Threat Discharge to Surface Waters Permit	WTP, RWPS, and raw and treated water pipelines
	NPDES Industrial Storm Water Permit	WTP
State Historic Preservation Office	Compliance with Sections 5024 and 5024.5 of the California Public Resources Code.	WTP, RWPS, and raw and treated water pipelines
Agency	Type of Approval	Project Component
California Department of Public Health	Domestic Water Supply Permit Amendment	WTP
Local/Other Agencies		
Monterey Bay Unified Air Pollution Control District	Authority to Construct	WTP, RWPS, and raw and treated water pipelines
	Permit to Operate	RWPS and WTP
San Benito County	Grading Permit	RWPS, WTP, and raw and treated water pipelines

14.2.1 Federal

The connection of the raw water pipeline to the Hollister Conduit requires coordination with the United States Bureau of Reclamation (USBR), the facility owner.

14.2.2 State

14.2.2.1 California Clean Air Act

Under the California Clean Air Act (CCAA), patterned after the federal CAA, areas have been designated as attainment or nonattainment with respect to the state standards. The Project area is nonattainment for particulates (PM_{10} and $PM_{2.5}$) and ozone. Responsibility for meeting California's standards lays with California Air Resources Board (CARB) and local air pollution control districts such as the Monterey Bay Unified Air Pollution Control District (APCD), which covers the Project area.

14.2.2.2 Porter-Cologne Water Quality Control Act

The Porter-Cologne Act (Division 7 of the California Water Code) provides the basis for water quality regulation within California and defines water quality objectives as the limits or levels of water constituents that are established for reasonable protection of beneficial uses. The SWRCB administers water rights, water pollution control, and water quality functions throughout California, while the Central Coast RWQCB conducts planning, permitting, and enforcement activities. The Porter-Cologne Act requires the RWQCB to establish water quality objectives, while acknowledging that water quality may be changed to some degree without unreasonably affecting beneficial uses. Beneficial uses, together with the corresponding water quality objectives, are defined as standards, per federal regulations. Therefore, the regional plans form the regulatory references for meeting state and federal requirements for water quality control. Changes in water quality are only allowed if the change is consistent with the maximum beneficial use of the state, does not unreasonably affect the present or anticipated beneficial uses, and does not result in water quality less than that prescribed in the water quality control plans.

14.2.2.3 NPDES Stormwater Construction Permit

The Central Coast RWQCB administers the NPDES stormwater permitting program in the Central Coast region. Construction activities disturbing one acre or more of land are subject to the permitting requirements of the NPDES General Permit for Discharges of Storm Water Runoff Associated with Construction Activity (General Storm Water Construction Permit). A Notice of Intent to the RWQCB is needed for the WTP to be covered by the General Construction Permit prior to the beginning of construction. The General Construction Permit requires the pre-construction preparation and implementation of a SWPPP.

14.2.2.4 NPDES Industrial Activities Stormwater Permit

The Central Coast RWQCB administers the NPDES stormwater permitting program in the Central Coast region. The regulations require that storm water associated with industrial activity (storm water) that discharges either directly to surface waters or indirectly through municipal separate storm sewers must be regulated by an NPDES permit. A Notice of Intent to the RWQCB is needed for the WTP to be covered by the General Industrial Activities Permit prior to the beginning of operation. The General Industrial Activities Permit requires the preparation and implementation of a SWPPP.

14.2.2.5 California Public Resources Code Sections 5024 and 5024.5 (Cultural Resources)

The California Environmental Quality Act (CEQA) requires that public projects or private projects financed or approved by public agencies must assess the effects of the project on historical resources. CEQA also applies to effects on archaeological sites, which may be included among “historical resources” as defined by Guidelines Section 15064.5, subdivision (a), or, in the alternative, may be subject to the provisions of Public Resources Code, Section 21083.2, which governs review of “unique archaeological resources.”

Historical resources may generally include buildings, sites, structures, objects, or districts, each of which may have historical, architectural, archaeological, cultural, or scientific significance. Archaeological resources that are not “historical resources” according to the above definitions may be “unique archaeological resources” as defined in Public Resources Code, Section 21083.2, which also generally provides that “non-unique archaeological resources” do not receive any protection under CEQA.

14.2.2.6 Domestic Water Supply Permit Amendment

The California Department of Public Health amends existing water supply permits, pursuant to the requirements of the California Health and Safety Code, Division 104, Part 12, Chapter 4 (California Safe Drinking Water Act), Article 7, Section 116550.

The City of Hollister and the SSCWD will need to amend their respective Domestic Water Supply Permits to include treated water supply from the West Hills WTP. The Owner of the West Hills WTP will need to obtain CDPH approval to operate the new plant and supply treated surface water. The Domestic Water Supply Permit will include a one-page form and supporting documents such as plans, specifications and an operations manual. It is recommended that the Owner meet and consult with CDPH on a regular basis over the course of the project design phases to address any CDPH concerns in advance.

14.2.3 Local

14.2.3.1 Air Permits - Authority to Construct/Permit to Operate

The Monterey Bay Unified APCD is the primary local agency responsible for protecting human health and property from the harmful effects of air pollution in the county's of San Benito, Monterey, and Santa Cruz, and has jurisdiction over most stationary source air quality matters.

The Monterey AQMD is responsible for developing attainment plans for inclusion in California's State Implementation Plan (SIP), as well as establishing and enforcing air pollution control rules and regulations. The attainment plans must demonstrate compliance with federal and State ambient air quality standards, and must first be approved by CARB before inclusion into the SIP. The Monterey AQMD regulates, permits, and inspects stationary sources of air pollution and is required to regulate emissions associated with sources such as agricultural burning and industrial operations.

All criteria pollutants are a concern of the Monterey AQMD, and a project's air quality impacts are considered significant if they would violate any of the state air quality standards. Ozone precursors, particulate matter emissions, and toxic air contaminants are emphasized in the review of applications for an Authority to Construct/Permit to Operate.

14.2.3.2 San Benito County Grading Permit

The purpose of San Benito County's Grading Permit safeguard public health, property and general welfare by regulating grading, drainage and erosion control on private and public property and requiring grading, erosion, and drainage control plans, which prevent water pollution and sedimentation of the county's water resources.

14.2.4 Other Permitting Requirements

This Section is primarily focused on the federal, state, and local permit requirements that will affect the design and operation of the proposed WHWTP. However, various other regulations and permitting requirements will affect design, construction, and operation of the plant. Project-specific permitting requirements will depend on various design decisions and site conditions and, therefore, are not determinable at this time. A permitting action plan will be included in the scope of services for the final design phase, including listing of regulatory agencies and other permitting requirements for the project.

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EXHIBIT 7.11

RECYCLED WATER FACILITIES PLAN

USE AREA EVALUATION TECHNICAL MEMORANDUM, 2012

USE AREA EVALUATION

Recycled Water Facilities Plan ***February 20, 2012***

Introduction

The San Benito County Water District (SBCWD) and the City of Hollister (City) have been developing a recycled water program to implement the beneficial use of treated effluent from the City's Water Reclamation Facility (WRF). This technical memorandum is the second in a series of technical memoranda. The first, Basis of Planning, described the project background and presented the planning criteria that will be used in developing the Recycled Water Facilities Plan (Plan). The purpose of this memorandum is to present the evaluation of two potential use areas, including the Wright Road / Buena Vista area and the McCloskey Road area. The evaluation is based on the use area evaluation criteria presented in the Basis of Planning memorandum, which included three non-economic criteria and a criterion for cost effectiveness. Thus, recycled water program costs are a key focus of this memorandum.

Background

The following subsections describe the projected recycled water availability and the use areas which will be evaluated later in this TM. As benchmarks over time, 2015 represents the initial year of proposed recycled water deliveries under Phase IIA and 2023 represents the end of the planning horizon, consistent with the Hollister Urban Area Water and Wastewater Master Plan.

Recycled Water Supply

As described in the Coordinated Water Supply and Treatment Plan (HDR 2010), development in the Hollister Urban Area has slowed due to the slowdown in the economy. As a result, the demand for water has actually decreased in recent years, from an estimated 7,300 AFY in 2007 to an estimated 6,200 AFY in 2010. Growth rates in the near term (through 2018) are expected to remain low compared to previous estimates. Consequently, the demand for potable water will not grow as quickly as expected, and thus, the supply of available recycled water will also be lower than expected.

Revised estimates of annually available recycled water supply, based on slower growth rates, are shown in Figure 1. In addition to the projected recycled water supply, Figure 1 also illustrates the capacity of the City's percolation basins, which represents an alternative method of disposal. While the City can percolate up to the amount shown, the shaded area between the inflow and total percolation capacity in Figure 1 represents the volume of recycled water that exceeds percolation capacity and thus, which must be used through a combination of irrigation at the City's Riverside Park, sprayfield irrigation, and agricultural use. It is notable that the inflow is not expected to exceed the percolation capacity until 2019. Irrigation at the City's Riverside Park is estimated to be approximately 138 AFY.

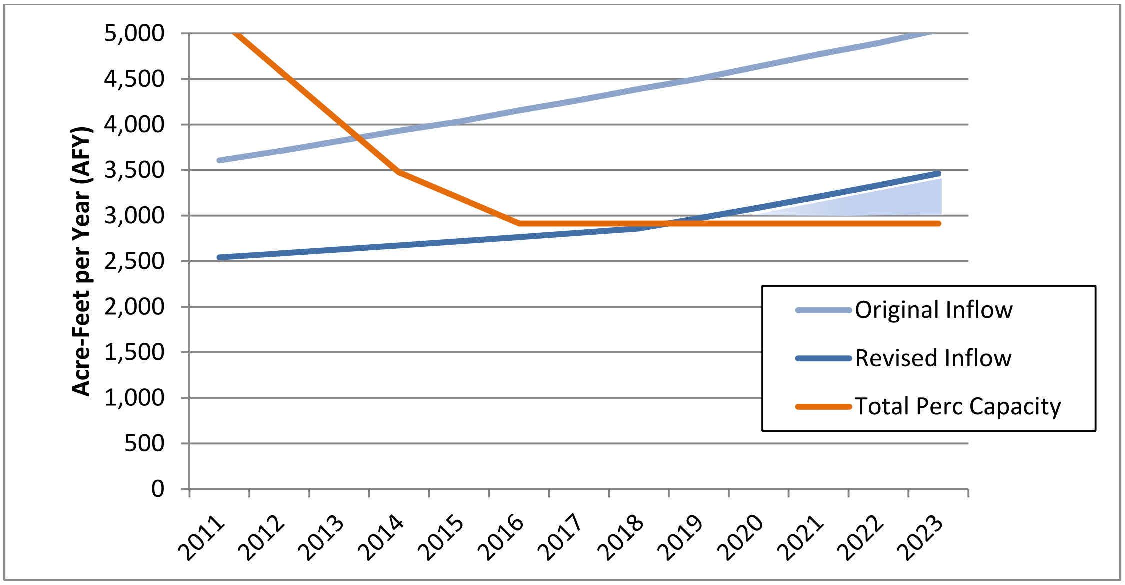


Figure 1. Annual Recycled Water Supply Forecast[1, 2]

The seasonally available recycled water supply depends on the amount of wastewater treated at the WRF, as well as percolation, rainfall, and evaporation in the recycled water seasonal storage basins if water is stored in the wet season for later use in the dry season. Figure 2 illustrates the projected variation in monthly recycled water supply for 2015 and 2023.

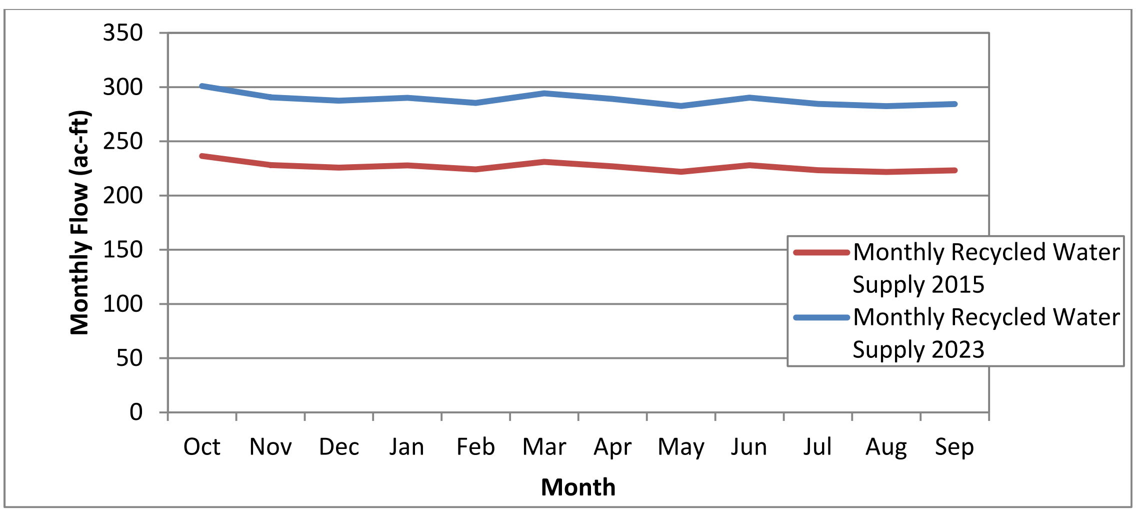


Figure 2. Monthly Recycled Water Supply

As shown in Figure 2, the recycled water availability averages approximately 225 ac-ft per month in 2015 and 290 ac-ft per month in 2023. Should the demand for recycled water exceed

[1] Projected growth rates and use rates are based on data provided by the City (Steve Wittry, March 2007).
[2] As stated in the City's Master Reclamation Requirements, Order No. R3-2008-0069, percolation at the Industrial Wastewater Treatment Plant (IWTP) shall be reduced over time and domestic wastewater is prohibited beyond December 31, 2015, in addition, percolation at the WRF basins must be reduced to a maximum of approximately 2900 AFY (2.6 mgd on an average annual basis).

these amounts, seasonal storage could be utilized to increase the available monthly supply. The current capacity of the City's seasonal storage reservoir is approximately 900 ac-ft; however, the reservoir is unlined, thus, as previously noted, there would be losses due to percolation.

Figure 3 illustrates the diurnal flows projected in July (peak growing season) for both 2015 and 2023. There are a number of options to consider if the peak daily demand for recycled water exceeds the available supply, including the use of buffering capacity (e.g., drawing from percolation ponds for peak demand storage) at the WRF, scheduling deliveries of recycled water to agricultural irrigators when greater hourly recycled water supply is available, or asking that irrigators use other existing sources of supply (e.g., existing wells) to cover peak-hour increments beyond recycled water supply limits.

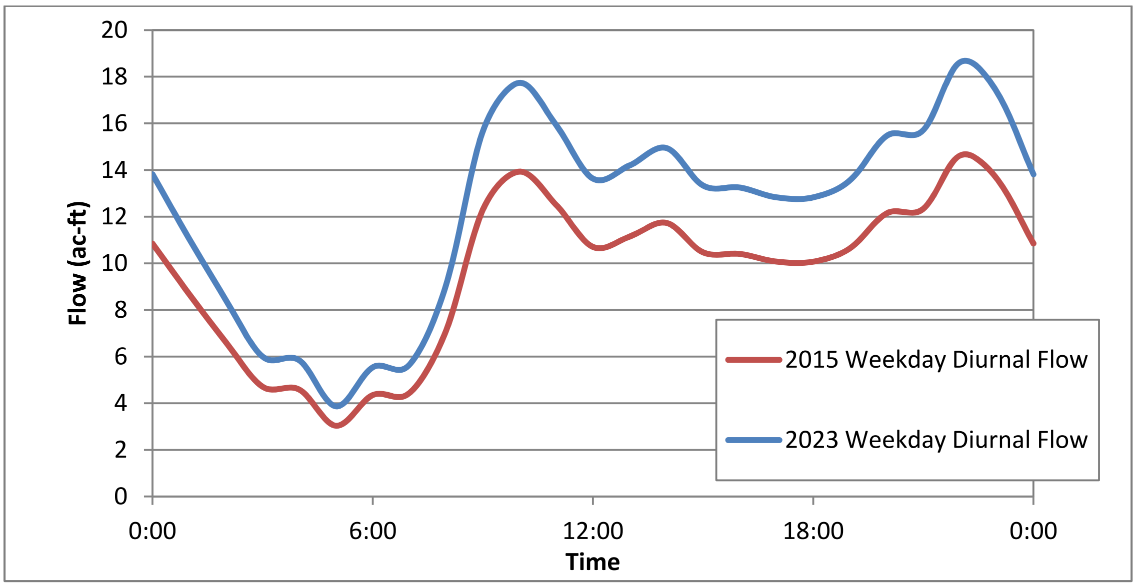


Figure 3. Diurnal Recycled Water Supply

Use Area Overview

As described in the previous subsection, the available supply of recycled water is expected to be lower than previously estimated; therefore it is appropriate to revisit the recommended use area. As described in the Recycled Water Feasibility Study (RWFS), Phase IIA was intended to include the area along both Wright Road and McCloskey Road. As shown on Figure 4, the Phase I pipeline constructed to convey water to the airport sprayfields is located in Wright Road. In order to serve water along the McCloskey Road corridor, the pipeline would need to be extended along McCloskey Road eastward toward Fairview Road.

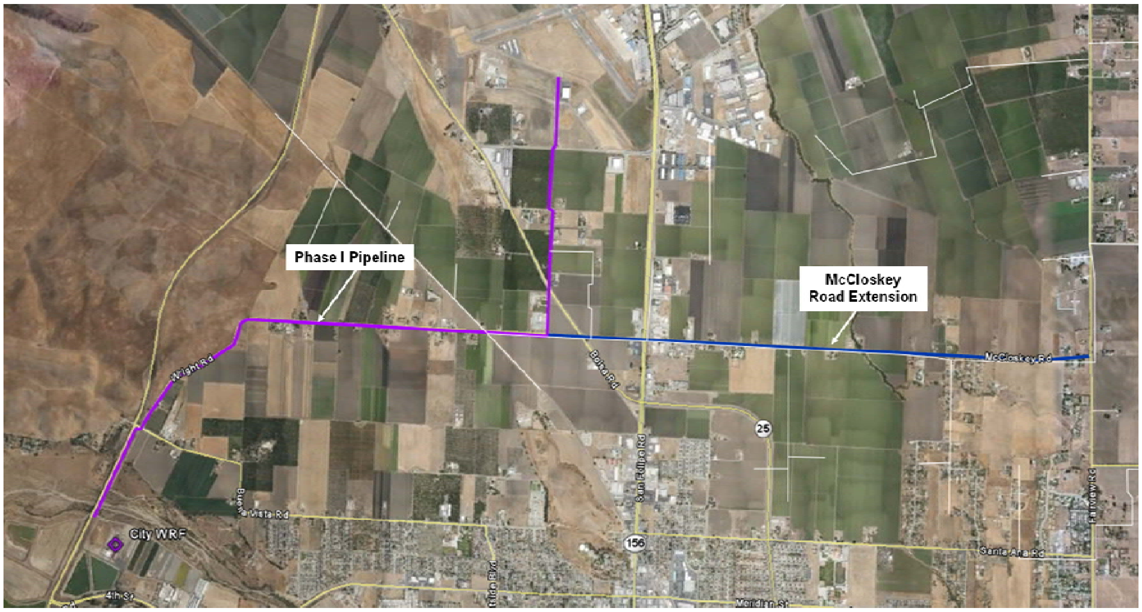


Figure 4. Recycled Water Phase I Facilities

Considering that existing Phase I infrastructure could be used to deliver recycled water to adjacent areas along the pipeline and that additional pipeline construction would be required to serve the McCloskey area, these two areas were identified as separate use areas for further evaluation. The two areas, referred to as 1) Wright Road / Buena Vista and 2) McCloskey Road, are shown in Figure 5.

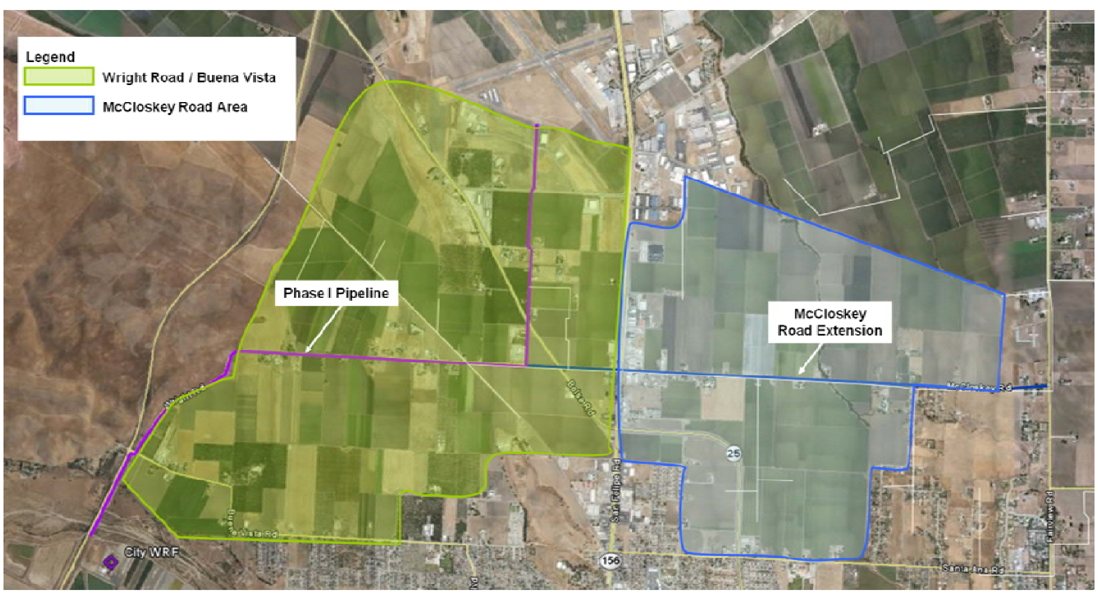


Figure 5. Phase IIA Potential Use Areas

Non-Economic Evaluation

As described in the Basis of Planning memorandum, there are three non-economic evaluation criteria:

- **Grower Acceptance.** Grower acceptance is a relative term that accounts for a proposed use area's (1) existing sources of water supply and (2) the cost associated with recycled water delivery relative to current CVP and groundwater unit costs. It is anticipated that proposed use areas with limited or no existing sources of supply would be rated higher with respect to this criterion over use areas with existing groundwater and/or CVP sources provided that unit costs of recycled water are competitive with current CVP costs.
- **Potential for Long-Term and Beneficial Use.** Potential use areas deemed to represent long-term demand and/or provide the flexibility for cost-effective system expansion will be considered advantageous. Proposed use areas associated with future land-use designations that will require termination of recycled water service in the future will be considered disadvantaged.
- **Regional Balance of Water Resources.** The selection of specific use areas will result in the ability to manage and/or influence both groundwater levels and quality.

The two potential Phase IIA use areas were considered with respect to the evaluation criteria. The results are summarized in Table 1 and described in the subsections below.

Table 1. Phase IIA Use Area Non-Economic Evaluation

Use Area	Grower Acceptance	Potential for Long-Term and Beneficial Use	Regional Balance of Water Resources
Wright Road / Buena Vista	High	Medium	High
McCloskey Road	Low	Medium	High

As indicated in Table 1, the Wright Road / Buena Vista area appears to be preferred over the McCloskey Road area with respect to non-economic evaluation criteria.

Grower Acceptance

Grower acceptance is anticipated to be greater in the Wright Road / Buena Vista area because many of the parcels in that area only have access to groundwater as a source of irrigation supply. Furthermore, the groundwater in that area is known to have very high TDS levels (greater than 1200 mg/L) and there are also concerns about the concentration of boron in the groundwater in that area, which can be problematic for certain orchard crops.

Grower acceptance in the McCloskey Road corridor is expected to be lower because the parcels in this area have access to CVP water as well as groundwater.

Potential for Long-Term and Beneficial Use

The use areas were ranked the same with respect to this criterion. Both use areas have areas within them that have been identified for rural residential development in the future. However, as described in the Master Plan (HDR 2008), development is not expected within the planning horizon.

Regional Balance of Water Resources

The McCloskey area was ranked high with respect to providing a regional balance of water resources because the use of recycled water, a local supply, would replace imported CVP water. By using recycled water, the overall quantity of water imported into the basin, and its associated salt load, would be reduced. Conversely, the use of recycled water in the Wright Road / Buena Vista area would not offset the use of imported CVP water. However, the use of recycled water in the area would provide a benefit to the users there, who have not had access to the high quality CVP water (from which the recycled water will largely be derived). Therefore, the Wright Road / Buena Vista area was also ranked high with respect to balancing regional water resources because the use of recycled water in that area expands the beneficiaries of imported CVP water.

Economic Evaluation

The economic evaluation includes both capital and operations and maintenance (O&M) costs. As will be shown in the following subsections, some costs are variable, while others are fixed. Thus, the cost per acre-foot is provided for the range of future demand scenarios.

The costs presented in the following sections will be further examined as specific use sites are identified and demands are quantified. The costs presented in this section are intended to provide enough information to determine 1) which potential use area is economically preferred, and 2) whether the project can be cost competitive with current rates for CVP water.

Capital Cost Evaluation

The cost of infrastructure required to provide recycled water to Phase II customers includes turnouts from the transmission main, recovery of the over-sizing of the Phase I pipeline and, if necessary, additional pipelines and storage. For the purpose of this analysis, it was assumed that users in the Wright Road / Buena Vista area could be served directly from the Phase I pipeline, while the McCloskey Road area would require a new pipeline to be constructed in McCloskey Road. Based on the actual parcels to be served, additional pipelines may be needed.

The cost of recovery for the construction of capital facilities was estimated over twenty years at a discount rate of 3%. These facilities are described in the following subsections. Because of its sensitivity, each facility's recovery cost is determined for a range of demand values, such that it can be expressed in dollars per acre-feet. This range runs from 650 AFY, based on low end demand projections, up to 2,000 AFY, based on higher demand projections through the planning period.

Turnouts

It is assumed that one or two new turnouts will be needed to provide recycled water to customers depending on recycled water demand and location of use areas and their proximity to one another. A summary of the cost per AF of recycled water produced is provided in Table 2.

Table 2. Turnout Cost per AFY of Recycled Water

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Number of Turnouts	--	2	2	2	2
Total Cost	$	32,000	32,000	32,000	32,000
Annual Cost Recovered	$/yr	2,151	2,151	2,151	2,151
Cost Per AF	$/AF	3	2	1	1

Recovery of Phase I Pipeline Over-Sizing

The existing Phase I pipeline was oversized to provide additional capacity to serve future recycled water customers. The oversized pipe allows for an additional 4,028 AFY in transmission capacity. The difference in cost associated with upsizing the pipeline from 14-inches to 20-inches in diameter was $829,730 based on the contractors bid for both pipe sizes which equates to an annual recovery cost of $55,771 per year. A summary of the cost per AFY of recycled water is provided in Table 3.

Table 3. Over-Sizing Recovery Cost per AFY of Recycled Water Produced

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Total Cost Recovery	$	829,730	829,730	829,730	829,730
Annual Recovery Cost	$/yr	55,771	55,771	55,771	55,771
Cost Per AF	$/AF	86	56	37	28

Pipeline Extension – Along McCloskey Road to Fairview Road

To provide recycled water to customers along McCloskey Road, shown in Figure 3, the Phase I pipeline would need to be extended along McCloskey Road toward Fairview Road, as shown in Figure 6.

Figure 6. Pipeline Extension along McCloskey Road to Fairview Road

The pipeline cost is based on a 20-inch diameter pipeline. A summary of the cost per AF of recycled water produced is provided in Table 4.

Table 4. McCloskey Road Pipeline Extension Cost per AF of Recycled Water

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Pipeline Length	lf	13,210	13,210	13,210	13,210
Pipeline Diameter	in	20	20	20	20
Capital Cost	$1,000	3,675	3,675	3,675	3,675
Annual Recovery Cost	$/yr	244,908	244,908	244,908	244,908
Cost Per AF	$/AF	377	245	163	122

Summary of Capital Costs

Table 5 provides a summary of the capital costs described above.

Table 5. Summary of Capital Costs

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Turnouts	$/AF	3	2	1	1
Phase I Pipe Over-size	$/AF	86	56	37	28
Subtotal	$/AF	89	58	39	29
McCloskey Pipeline Extension	$/AF	377	245	163	122
Cost Per AF	$/AF	466	303	202	151

Recycled Water Delivery Costs

The costs associated with the normal operation of the facilities necessary to deliver recycled water include pumping power, hypochlorite for disinfection, and monitoring. Each is described below.

Power Consumption

The City is billed energy and demand charges based on a PG&E E19P rate schedule. For each of these charges, there are separate rates for the summer and winter in which power consumption is charged based on time of use. Due to the complexity of the charges, a weighted rate was estimated based on historical rates and use at the WRF. The weighted rate, $0.18 per kWh, was then used to estimate power costs associated with the operation of the recycled water pump station at the WRF.

The annual power usage was estimated based on a discharge pressure of 85 psi and a typical pump efficiency of 75 percent. A summary of the total power costs and cost per AFY of recycled water produced is provided in Table 6.

Table 6. Power Cost per AF of Recycled Water

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Power Unit Cost[1]	$/kWh	0.18	0.18	0.18	0.18
Power Usage[2]	kWh/yr	233,137	358,672	538,008	717,345
Total Power Cost	$/yr	41,965	64,561	96,842	129,122
Cost Per AF	$/AF	65	65	65	65

1) Unit cost is a weighted rate based on historical rates and use at the City's WRF.
2) Pump station power usage is based on an assumed discharge pressure of 85 psi and typical pump efficiency of 75 percent.

Disinfection Chemicals

Hypochlorite dosing provides disinfection of the recycled water at the WRF. Hypochlorite is fed at a rate of 8 mg/L at the WRF to achieve the necessary minimum chlorine residual of 5 mg/L. Based on a hypochlorite unit cost of $1.00 per gallon, the estimated cost to disinfect one acre-foot of recycled water is about $19.06, as shown in Table 7.

Table 7. Disinfection Cost per AF of Recycled Water

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Hypochlorite Unit Cost	$/gal	1.00	1.00	1.00	1.00
Hypochlorite Use	gal/yr	12,392	19,064	28,596	38,128
Total Hypochlorite Cost	$/yr	12,392	19,064	28,596	38,128
Cost Per AF	$/AF	19	19	19	19

Monitoring

Monitoring costs include water quality, groundwater, and nutrient management plan monitoring. Based on initial discussions with the Regional Water Quality Control Board (RWQCB), the preliminary water quality monitoring program summarized in Table 8 was developed. Monitoring would occur at one or two turnouts. Monitoring at the wells would occur at four existing wells (MW-11, MW-12, MW-19, and MW-46) and three future wells.

Table 8. Proposed Water Quality Monitoring Program

Constituents	Water Reclamation Facility		Turnouts[2,3]	Monitoring Wells[1]
	Recycled Water[1]	Storage Pond		
BOD	Weekly			
Chloride			Quarterly[4]	Twice Annually
Chlorine Residual	Continuous	Continuous[2]	Weekly[2]	
Coliform, Fecal	Weekly[2]		Bi-weekly[2]	
Coliform, Total	Weekly		Bi-weekly[2]	
Generic E.Coli	Weekly[2]		Bi-weekly[2]	
Haloacetic Acids			Quarterly[4]	
Metals	Twice Annually			Twice Annually
Nitrogen, Ammonia	Weekly			
Nitrogen, Nitrate	Weekly			Quarterly[2]
Nitrogen, Nitrite	Weekly			
Nitrogen, TKN	Quarterly			
Nitrogen, Total	Quarterly			Quarterly[2]
pH	Weekly	Weekly[2]	Weekly[2]	Twice Annually
Sodium			Quarterly[4]	Twice Annually
Specific Conductance	Continuous	Continuous[2]	Weekly[2]	Twice Annually
Sulfate				Twice Annually
Total Dissolved Solids	Weekly[2]	Weekly[2]		Quarterly[2]
Total Suspended Solids	Weekly			
Tri-halomethanes			Quarterly[4]	
Turbidity	Continuous			

1) Currently monitored by the SBCWD or City, except as noted.
2) New monitoring efforts.
3) Monitoring at two turnouts.
4) Monitoring at one turnout.

As noted above, groundwater monitoring includes the construction of three additional monitoring wells, which for the sake of simplicity, has been included with the O&M costs presented in Table 9. The cost of recovery for the construction of the monitoring wells was estimated over twenty years at an interest rate of 3%. Nutrient management plan monitoring includes soil sampling, laboratory analysis, and developing an analysis report. A summary of monitoring costs is provided in Table 9.

Table 9. Monitoring Cost per AFY of Recycled Water

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Water Quality Monitoring	$/yr	26,377	26,377	26,377	26,377
Groundwater Monitoring	$/yr	15,025	15,025	15,025	15,025
Nutrient Management Plan	$/yr	21,528	21,528	43,056	43,056
Total Monitoring Cost	$/yr	62,930	62,930	84,458	84,458
Cost Per AF	$/AF	97	63	56	42

Summary of Recycled Water Delivery Costs

Table 10 summarizes the O&M costs associated with delivering recycled water as presented in the preceding sections. It should be noted that these costs do not include labor.

Table 10. Summary of O&M Cost per AF of Recycled Water

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Power	$/AF	65	65	65	65
Disinfection	$/AF	19	19	19	19
Monitoring Costs	$/AF	97	63	56	42
O&M Cost per AF	$/AF	180	146	140	126

Summary of Economic Evaluation

The total capital and O&M unit costs presented in the previous sections are summarized for the two potential use areas in Table 11 and Table 12, respectively.

Table 11. Total Costs for the Wright Road / Buena Vista Area

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Capital Costs	$/AF	89	58	39	29
O&M Costs	$/AF	180	146	140	126
Total Costs	$/AF	269	204	178	155

Table 12. Total Costs for the McCloskey Road Area

Component	Units	Recycled Water Demand (AFY)			
		650	1,000	1,500	2,000
Capital Costs	$/AF	466	303	202	151
O&M Costs	$/AF	180	146	140	126
Total Costs	$/A	646	449	342	277

As previously described, it is also important to determine whether the cost of the recycled water program is cost competitive with the cost of CVP supply. The cost of CVP water based on the current rates for water year 2011-2012, includes a water charge of $155 per AF for agricultural customers and a power charge of $51.25 per AF (for Subsystem 9L). Therefore, the cost of CVP water is $206 per acre-foot. Based on the values presented in Table 11, it is clear that the cost of serving recycled water in the Wright Road / Buena Vista area could be cost competitive with current CVP rates, and as more recycled water is used, the unit cost decreases because the fixed cost associated with capital recovery and monitoring are spread over a larger quantity, as illustrated in Figure 7.

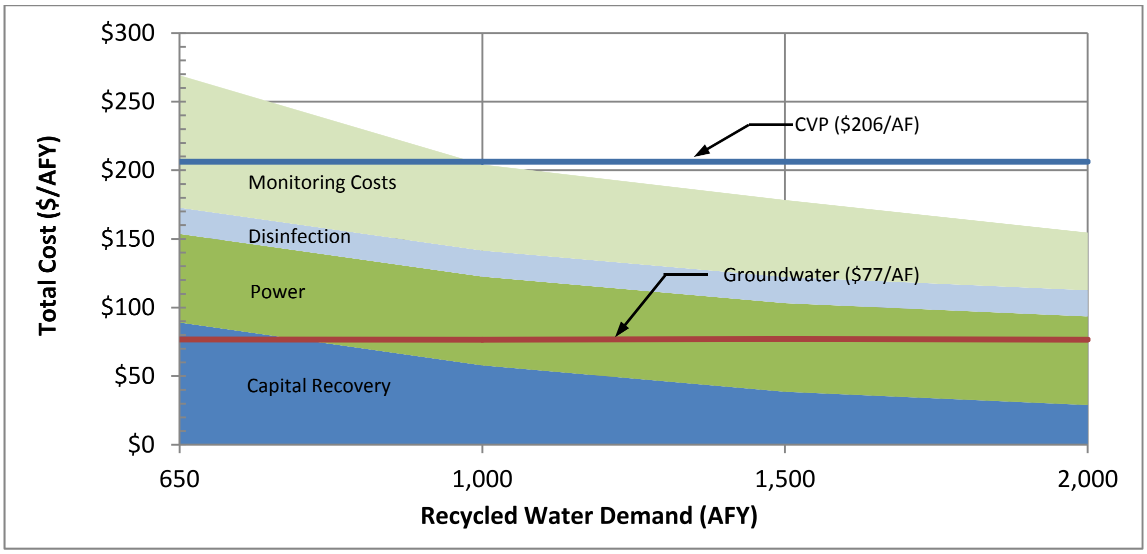


Figure 7. Recycled Water Production Costs vs. CVP Water and Groundwater

The cost of serving recycled water to the McCloskey Road Area is more than double the cost to serve the Wright Road/Buena Vista area. As mentioned in the non-economic analysis the McCloskey area has better access to the CVP water. Therefore, the McCloskey Road Area Extension is the less attractive option.

Because many of the existing irrigators in the Wright Road / Buena Vista area use groundwater, the cost of using groundwater was also estimated for comparison purposes. The cost to pump groundwater includes a groundwater charge of $2.50 per acre-foot for agricultural customers based on the current rates for water year 2011-2012. The depth to groundwater in the Wright Road / Buena Vista area is about 100 ft based on the *Annual Groundwater Report for Water Year 2010*. Power requirements to pump groundwater from this depth were estimated based on the $0.18/kWh estimated above and a system pressure of 65 psi (similar to CVP). The resulting cost to pump groundwater is approximately $77 per acre-foot, which is much lower than the cost of recycled water (see Figure 7). However, as mentioned in the non-economic analysis, the groundwater has high concentrations of TDS and trace amounts of boron, which negatively affect crop output. Despite the lower cost of water, the crop yield for fields irrigated with groundwater is less than the yield of fields irrigated with high quality water. In particular, cash crops such as leafy greens, asparagus, peppers and tomatoes that are irrigated with high quality water experience better germination, faster transplant establishment, and higher yield, which in turn increases revenue.

As shown in , as the demand for recycled water increases, the unit cost per acre-foot declines. In order to be cost competitive with CVP water without subsidizing the recycled water program, the demand must reach at least 1,000 AFY.

The estimated annual supply of recycled water based on current flows to the City's WRF is approximately 2500 AFY (based on an inflow of 2.23 mgd). However, much of that supply is created during the winter season, when there is little to no agricultural demand. While some recycled water can be stored in the existing seasonal storage reservoir and carried over for the irrigation season, the reservoir is unlined and has a limited capacity. Therefore, as demand grows beyond that which can be supplied from daily flows to the City's WRF, improvements to existing and/or additional seasonal storage may be desired. The additional capital cost associated with that improvement has not been included, but will be part of the next steps if needed.

APPENDIX

EXHIBIT 7.12

PSMCSD TANK INSPECTION REPORT, 2013

Inland Potable Services, Inc.

16297 E. Crestline Lane
Centennial, CO 80015
Phone: 303-400-4220
Fax: 303-400-4215

Inspection Report for

Pajaro/Sunny Mesa CSD
Royal Oaks, CA

600KG Steel On-Grade Tank

Date Completed: February 2, 2013

Commercial Dive Team:

Diver –James Bingham
Dive Controller –Jason Gardner
Tender –Jeff Roberts

Scope of Work:

Our team completed sediment removal using underwater vacuum equipment. Sediment depth averaging 1/8 inch (iron & manganese) was removed from tank floor. When the cleaning process was finished, a full visual inspection was performed of the tank interior and all interior fixtures. The team also performed a full visual inspection of the tank exterior and all attached fixtures. The details of the inspection findings are included in the report below.

Summary of the Inspection:

Exterior Inspection

1. There was good access to the tank. (In a gated area)
2. The ladder was found secure, OSHA approved and in good condition with biological growth, minor de-lamination, heavy oxidation and less than 1% surface corrosion noted.
3. The roof was found in good condition with heavy biological growth & oxidation, low spots and 1% surface corrosion noted.
4. The hatch was found locked with no gasket present and in poor condition with biological growth, de-alloying, heavy oxidation, de-lamination and 33% surface corrosion noted.
5. The walls were found in good condition with biological growth and 1% surface corrosion noted.
6. The vent was found in fair condition with heavy oxidation, minor de-lamination and 33% surface corrosion noted and a screen in place.
7. The manways were found secure and in good condition with biological growth, de-alloying, de-lamination and 1% surface corrosion noted.

Interior Inspection

1. The inlet and outlet were found in good condition with heavy staining and less than 1% surface corrosion noted.
2. The ladder, overflow, support column and floor were found in fair condition with heavy staining, rust nodules, blistering, pitting and surface corrosion noted.
3. The manways were found in good condition with heavy staining & blistering and less than 1% surface corrosion noted.
4. The interior walls were found in fair condition with heavy staining, micro blisters, pitting and 1% surface corrosion noted.
5. The interior roof was found in good condition with concentrated cell corrosion, corrosive staining, de-alloying and 2% surface corrosion noted. The support beams were also found to be warped.

Recommendations:

1. Install a gasket on the access hatch.
2. Schedule time for a blast and recoat.
3. Schedule time to clean and inspect every 3-5 years per AWWA recommendations.

Key

Excellent – Like new, no repairs needed
Good – Cosmetic problems, repair if utility wants
Fair – Minor problems, repairs needed
Poor – Major problems, fix now

Inland Potable Services, Inc.

Exterior Inspection Report

Access Ladder Condition

Ladder Type: Steel
Coating Condition: Fair
Corrosion Present? Y ☒ N ☐
Seams/Welds Condition: Excellent
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Stand Off Supports Condition: Good
Safety Climb Type: Cage
Safety Climb Condition: Excellent
Is Top Of Tank Easily Accessible? Y ☒ N ☐
Is The Ladder and Safety Climb **OSHA** Approved? Y ☒ N ☐

Summary: The ladder was found secure, OSHA approved and in good condition with biological growth, minor de-lamination, heavy oxidation and less than 1% surface corrosion noted.

Roof Condition

Coating Condition: Good
Corrosion Present? Y ☒ N ☐
Percentage: 1%
Seams/Welds Condition: Excellent
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Low Spots Present? Y ☒ N ☐
Holes in Roof? Y ☐ N ☒
Cathodic Protection Plates Present? Y ☒ N ☐
Sealed Edges: Y ☒ N ☐ N/A ☐
Loose Plates? Y ☐ N ☒ N/A ☐
Missing Plates? Y ☐ N ☒ N/A ☐

Summary: The roof was found in good condition with heavy biological growth & oxidation, low spots and 1% surface corrosion noted.

Access Hatch Condition

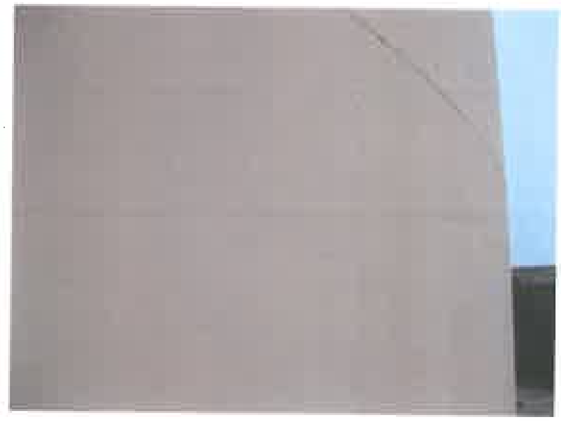
Coating Condition: Poor
Corrosion Present: Y ☒ N ☐
Seams/Welds Condition: Good
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Hatch Size: 2 foot square
Hatch Locked? Y ☒ N ☐
Hinge Condition: Good
Gasket Present? Y ☐ N ☒
Intact? Y ☐ N ☐ N/A ☒
Insects, Dirt Or Debris Present Under Hatch? Y ☐ N ☒

Summary: The hatch was found locked with no gasket present and in poor condition with biological growth, de-alloying, heavy oxidation, de-lamination and 33% surface corrosion noted. Recommend a gasket.

Wall Panel Condition

Coating Condition: Good
Corrosion Present? Y ☒ N ☐
Percentage: 1%
Seams/Welds Condition: Excellent
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Dents Present? Y ☐ N ☒
Holes Present? Y ☐ N ☒

Summary: The walls were found in good condition with biological growth and 1% surface corrosion noted.



Vent Condition

Coating Condition: Fair
Corrosion Present: Y ☒ N ☐
Percentage: 33%
Seams/Welds Condition:
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐

Screen in Place? Y ☒ N ☐
Condition: Good
All Openings Sealed? Y ☒ N ☐
Cap Condition: Good

Summary: The vent was found in fair condition with heavy oxidation, minor de-lamination and 33% surface corrosion noted and a screen in place.

Foundation Condition

Foundation Exposed? Y ☒ N ☐
Anchor Bolts Present? Y ☐ N ☒
Corrosion on Anchor Bolts Present? Y ☐ N ☐ N/A ☒
Anchor Bolts Loose? Y ☐ N ☐ N/A ☒

Cracking Noted In Foundation? Y ☐ N ☒
Spalling Noted? Y ☐ N ☒

Summary: The foundation was found in excellent condition.

Manway Condition	
Coating Condition: Both Fair Weld/Seam Condition: Both Excellent Corrosion Present? Y ☒ N ☐ Percentage: 1% Pitting Noted In Metal? Y ☐ N ☒ Depth: N/A Summary: The manways were found secure and in good condition with biological growth, de-alloying, de-lamination and 1% surface corrosion noted.	
	

Inland Potable Services, Inc.
Interior Inspection Report

<table>
<tr><th colspan="2">Inlet and Outlet Condition</th></tr>
<tr><td>Common Inlet/Outlet? Y ☐ N ☒ Location: N/A
If No:
Inlet Location: 4:30 o'clock
Coating Condition: Good
Weld/Seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: less than 1%
Pitting Noted In Metal? Y ☐ N ☒
Depth: N/A

Summary: The inlet was found in good condition with heavy staining and less than 1% surface corrosion noted.</td><td></td></tr>
<tr><td>Common Inlet/Outlet? Y ☐ N ☐ Location:
If No:
Outlet Location: 10:30 o'clock
Coating Condition: Good
Weld/Seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: less than 1%
Pitting Noted In Metal? Y ☐ N ☒
Depth: N/A

Summary: The outlet was found in good condition with heavy staining and less than 1% surface corrosion noted.</td><td></td></tr>
<tr><th colspan="2">Ladder Condition</th></tr>
<tr><td>Ladder Location: 12 o'clock
Coating Condition: Fair
Weld/Seam Condition: Fair
Supports Condition: Fair
Corrosion Present? Y ☒ N ☐
Percentage: 3%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The ladder was found in fair condition with heavy staining, rust nodules, heavy blistering, pitting and 3% surface corrosion noted. The cage was found in poor condition.</td><td></td></tr>
</table>

Manway Condition	
Manway Locations: 3 o'clock & 8 o'clock Coating Condition: Both Fair Weld/Seam Condition: Both Excellent Corrosion Present? Y ☒ N ☐ Percentage: less than 1% Pitting Noted In Metal? Y ☐ N ☒ Depth: N/A Summary: The manways were found in good condition with heavy staining & blistering and less than 1% surface corrosion noted.	
	

Overflow Condition	
Overflow Location: 9:30 o'clock Coating Condition: Fair Weld/Seam Condition: Excellent Corrosion Present? Y ☒ N ☐ Percentage: 1% Pitting Noted In Metal? Y ☒ N ☐ Depth: 1/8 inch Summary: The overflow was found in fair condition with heavy staining & blistering, corrosive staining, rust nodules, pitting and 1% surface corrosion noted.	

Wall Panel Condition

Coating Condition: Fair
Welds/seam Condition: Excellent
Corrosion Present On Panel? Y ☒ N ☐
Percentage: 1%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The interior walls were found in fair condition with heavy staining, micro blisters, pitting and 1% surface corrosion noted.

Roof Condition

Coating Condition: Fair
Welds/seam Condition: Good
Corrosion Present On Panels? Y ☒ N ☐
Percentage: 2%
Metal De-alloying Noted? Y ☒ N ☐
Percentage: less than 1%

Summary: The interior roof was found in good condition with concentrated cell corrosion, corrosive staining, de-alloying and 2% surface corrosion noted. The support beams were found to be warped.

Support Column Condition

Coating Condition: Fair
Welds/seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percent: 2%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

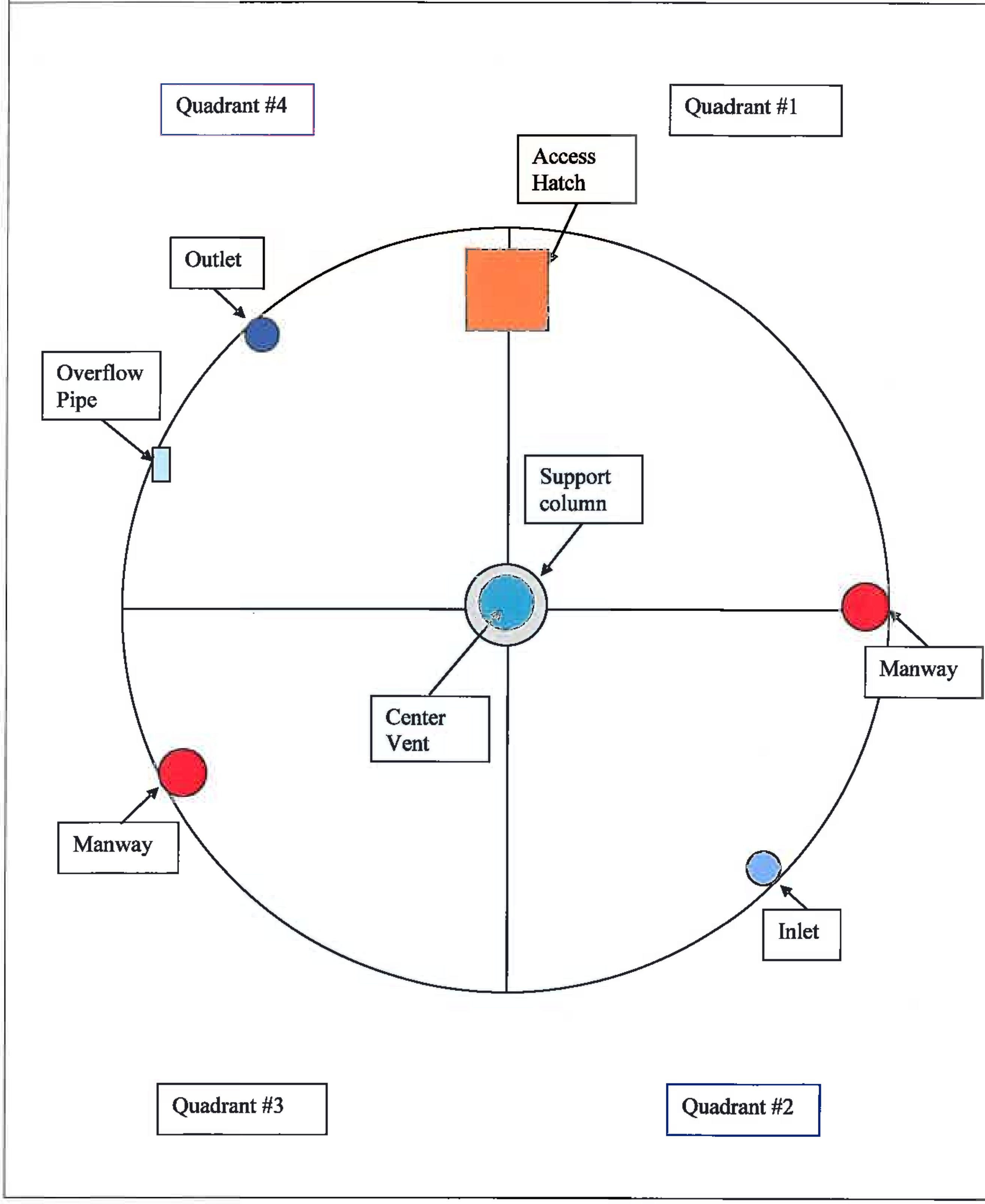
Summary: The support column was found secure and in fair condition with heavy staining, rust nodules, pitting and 2% surface corrosion noted.

Floor Condition

Coating Condition: Fair
Welds/seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: 2%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The floor was found in fair condition with heavy staining, rust nodules, blistering, pitting and 2% surface corrosion noted.

Tank Layout



Quadrant #4
Quadrant #1
Access Hatch
Outlet
Overflow Pipe
Support column
Manway
Center Vent
Manway
Inlet
Quadrant #3
Quadrant #2

EXHIBIT 7.13

PSMCSD PRELIMINARY ENGINEERING REPORT, 2013

Kennedy/Jenks Consultants

20 February 2013

REGISTERED PROFESSIONAL ENGINEER
THOMAS E. YEAGER
No. C 48014
Exp. 2-20-13
CIVIL
STATE OF CALIFORNIA
12-31-13

Memorandum

To: Don Rosa

From: Nick Panofsky

Reviewed by: Tom Yeager

Subject: Pajaro Sunny Mesa CSD – Proposition 84 Preliminary Engineering Report
K/J 985019*00

Introduction

The purpose of this memo is to describe alternatives for improving potable water storage for the Pajaro/Sunny Mesa Community Services District (P/SM CSD). This memo will examine two options for improving potable water storage.

Background

The Pajaro/Sunny Mesa Community Services District (P/SM CSD) was incorporated in 1992 and provides water service to the unincorporated communities of Pajaro and Sunny Mesa in Monterey County. Currently there are approximately 700 customers in these two service areas. These two service areas are geographically separated and each has their own water systems that are not interconnected.

The Pajaro system has a 1500 gpm well, one above ground 600,000 welded steel storage tank, and a booster pump system utilizing hydropneumatic tanks. The current maximum daily demand for the Pajaro system is approximately 500,000 gallons; the minimum fire flow storage requirements are 2 hours at 1500 gpm (180,000 gallons). Therefore, the storage in Pajaro is not adequate for fire flow and maximum daily demand as required by the California Department of Public Health (Department) and local fire codes.

In a Water System Inspection Report published by the Department, dated September 29th, 2010, several issues were identified. A copy of this report is included as Appendix A. These issues included the following observations and recommendations:

- The screen on the roof vent is broken and must be replaced to prevent possible contamination.

- The hatch interior, the top of the interior ladder and the roof vent are badly corroded and need to be cleaned and recoated per AWWA standards.

- It is strongly recommended by the Department that the tank interior be thoroughly inspected for any structural damage caused by the corrosion. If not repaired over time, this problem may result in a sanitary hazard.

Project Area

The P/SM CSD currently owns several adjoining parcels of land on Railroad Avenue, near the intersection of Railroad Avenue and Allison Road near Pajaro. The existing site has an existing surface elevation of approximately 30 feet. At this site, there is an existing 600,000 gallon potable water storage tank, a pump station, one active well, two hydropneumatic tanks, miscellaneous piping associated with the existing water system, and storage for the P/SM CSD's other maintenance activities.

The P/SM CSD has acquired additional land to the west of the site of their existing facilities. This land is undeveloped and will be used for a new water storage tank. . See Figure 1 for the location of the existing tank in relation to the proposed site of the new tank.

Geotechnical Considerations

A geotechnical study was prepared for the existing 600,000 gallon tank, but additional work for the new tank has not yet been performed. It is assumed that geotechnical conditions would be similar as the new tank site is next to the existing tank site

The existing soils at the site are expected to be significantly compressible due to a high water content and small soil particle size. Due to this condition, a wick drain system, similar to that installed for the existing 600,000 gallon tank would be provided to consolidate the soils below the new water storage tank to reduce the likelihood of long term settling.

Wick drains are prefabricated vertical drains installed to accelerate the consolidation of compressible soils. Wick drains consist of a geotextile filter-wrapped plastic strip with an extruded channel that allows water to drain from the soil as it consolidates under an applied surcharge load. If wick drains are required for this project, a wick drain system will be installed, then the new tank constructed immediately after. The tank will act as the surcharge load to consolidate the soils. When the soils have compressed sufficiently and the tank has settled to its final position, the final connection to the potable water system will be made.

A detailed geotechnical study will be completed prior to the start of construction to verify the site is suitable for a wick drain system and construction of additional potable water storage.

Existing Storage Tank

The existing 600,000 gallon welded steel water storage tank was constructed in the 1980s. An inspection of the existing storage tank was conducted on February 2nd, 2013 by Inland Potable Services, Inc. An inspection report was prepared as part of this inspection, and is included as Appendix B of this report. The inspection report documents the condition of the existing tank.

The results of this inspection indicate that the existing tank has been adversely impacted by corrosion. While in overall good condition, significant deficiencies were noted. The tank vents and hatches are corroded and there is corrosion on the interior and exterior of the tank, especially along the roof beams and the dollar plate where the roof beams are connected to the center support column. A rehabilitation project should be completed on this tank so that it can remain in service for an extended period of time. The entire tank should be sandblasted and recoated, structural steel repaired, and the degraded appurtenances should be repaired or replaced.

A budgetary cost estimate of $600,000 has been developed to repair and rehabilitate the existing 600,000 gallon water storage tank in accordance with the recommendations of the inspection report and Department recommendations. In addition, engineering and environmental costs are estimated at $140,000 for a total project cost of $750,000. These budgetary cost estimates are included in Appendix C. These budgetary cost estimates do not include an allowance for supplementary storage while the tank is being rehabilitated.

Additional Storage Requirements

The P/SM CSD currently relies on a single welded steel water storage tank for its water storage requirements. This design severely limits the operational flexibility of the system. If the existing tank is taken out of service for repairs or an emergency situation, there is no back-up storage for the system. As previously described, the existing storage tank is currently in need of repairs. In order to make these repairs the existing storage tank would need to be out of service for 6-12 weeks.

There are two options for improving the water storage capacity for the system. Option A would construct a second 600,000 gallon welded steel tank, and then rehabilitate the existing 600,000 gallon tank. Option B would construct a new 1,200,000 gallon concrete storage tank, and then demolish the existing 600,000 gallon tank.

Option A would provide operational redundancy, as well as additional water storage capacity. Option B would not provide the operational redundancy of two separate tanks, but would provide a design that could be serviced without taking the tank out of service since it is a concrete tank and not a welded steel tank, as well as additional water storage capacity.

Option A

Option A includes the construction of a second 600,000 gallon welded steel tank and rehabilitation of the existing 600,000 gallon welded steel tank. The existing tank would be rehabilitated as previously described after construction of the new tank was completed, and funds became available.

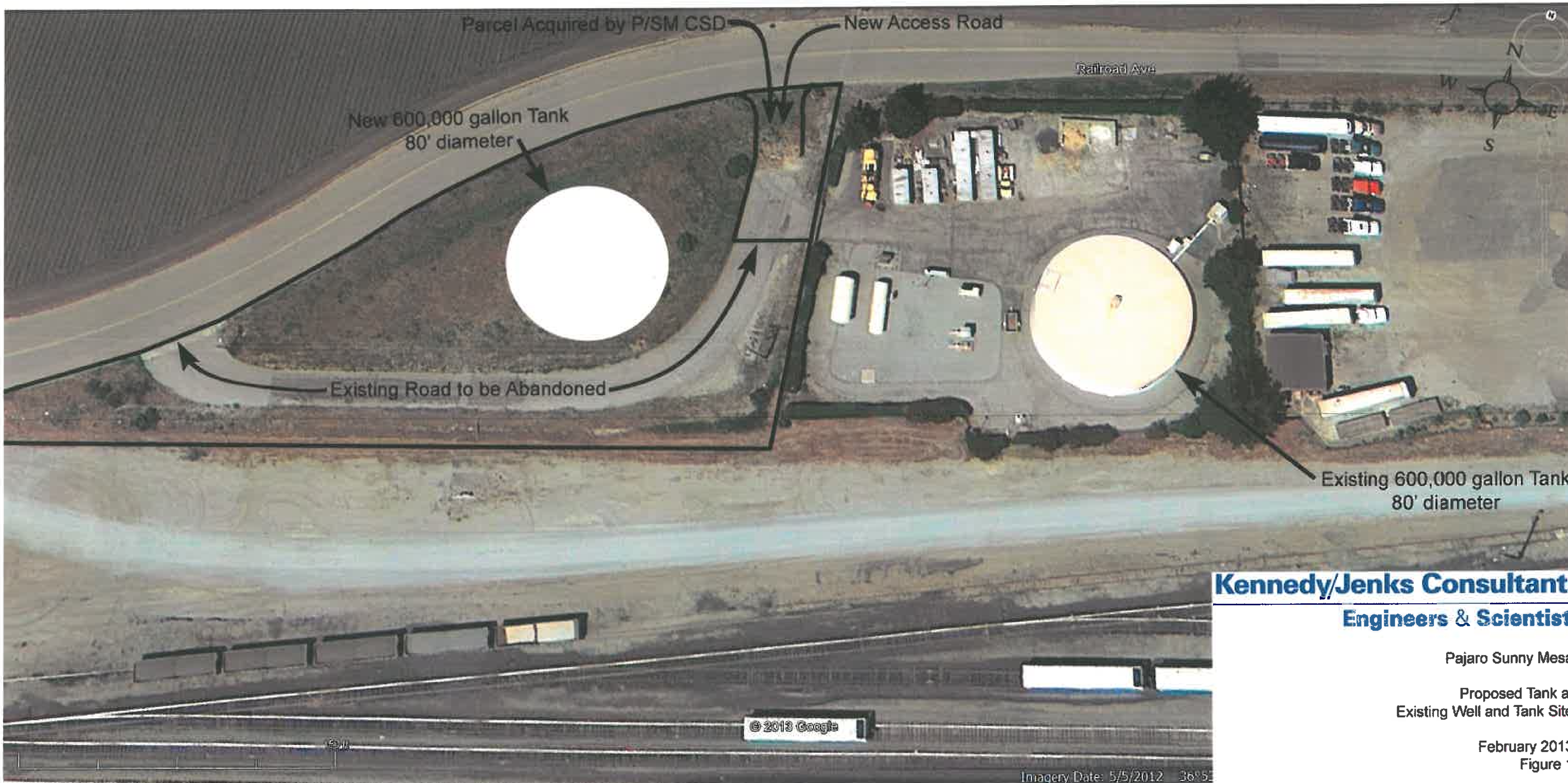
The new tank would be constructed in the undeveloped area adjacent to the site of the existing water storage tank. The new tank would be an 80 foot diameter tank, with an approximate height of 20 feet. The new tank would be equipped with typical appurtenances, including access ladders, access hatches, vents, etc., and would be suitable for potable water storage. This construction, including a wick drain system, but not including the rehabilitation of the existing storage tank has an expected construction cost of $1,400,000. In addition, environmental and engineering costs are estimated at $370,000 for a total project cost of $1,770,000. These budgetary cost estimates are included in Appendix C.

Option B

Option B includes the demolition of the existing 600,000 gallon welded steel tank and the construction of a new 1,200,000 gallon concrete storage tank. The existing tank would be demolished and sold for scrap. The new tank would be constructed in the undeveloped area adjacent to the site of the existing water storage tank. The new tank would be a 120 foot diameter tank, with an approximate height of 20 feet. The new tank would be equipped with typical appurtenances, including access ladders, access hatches, vents, etc., and would be suitable for potable water storage. This construction, including the wick drain system, has an expected construction cost of $1,900,000. In addition, environmental and engineering costs are estimated at $440,000 for a total project cost of $2,340,000. These budgetary cost estimates are included in Appendix C.

Recommended Option

Option A is the recommended alternative. Although the construction of a new tank, and rehabilitation of the existing tank has a slightly higher overall capital cost, Option A allows the construction to be divided into two phases. Construction and operation of a new tank during the first phase will provide for additional storage and provide the necessary redundancy so that the existing tank can be taken off line in to order to alleviate the potential sanitation issues raised by the California Department of Public Health. Rehabilitation of the existing tank during the second phase can proceed when funds are available.



Kennedy/Jenks Consultants

Engineers & Scientists

Pajaro Sunny Mesa

Proposed Tank at
Existing Well and Tank Site

February 2013
Figure 1

APPENDIX A

Water System Inspection Report

Water System Inspection Report
California Department of Public Health
Drinking Water Field Operations Branch
Monterey District

Water System: **Pajaro CSD**	Water System Number: **2710020**
Contact: **Joe Rosa**	Title: **General Manager**
Inspection Date: **July 12, 2010**	Inspector: **Shaminder Kler/Querube Moltrup**
Report Date: **September 29, 2010**	

Water System Information

System Location and Operation

The Pajaro Community Services District (Pajaro CSD) water system is located in the community of Pajaro in Monterey County just south of Watsonville. The system is a community water system supplying domestic water via 453 service connections to a permanent population of about 4,500 and maximum population of about 6,500 persons.

The system is owned by the Pajaro/Sunny Mesa Community Services District (P/SM CSD). The operation and management of the District is overseen by a Board of Directors. The members of the board are appointed by the Monterey District Supervisor and must reside within the District's service area. The water district meets with the board once per month to report on any specific issues. The District has a 1-year budget and a 5-year plan. Rates are set based on system operation (i.e., rates cover the cost to operate the water system and provide scheduled maintenance of system facilities). However, the rates do not include a source of revenue for funding large capital improvement projects. The Pajaro/Sunny Mesa Community Service District relies on Funding Programs like SRF to fund large projects. .

The system has been classified as a D2 for distribution system operation and a T1 for treatment facility operation. The system is operated by following three in-house certified operators:

Operator Name	Distribution Certification	Treatment Certification
Rodney Schmidt	D2	T1
Don Rosa	D2	T1
Elden Pierce	D2	T1

All the operators are also certified backflow prevention device testers.

Permit Status

The Department issued Permit No. 87-015 to Pajaro CSD on March 11, 1987. There have been no amendments to the original permit. The system previously used chlorine gas to disinfect the raw water at Well 02 but converted to chlorine tablets in October 2006. This change in treatment method requires an amendment to the permit. The system submitted a permit amendment application to modify treatment to use Trichlor tablets (Trichloroisocyanuric Acid ARDEN 90PT) on May 22, 2009. The Department is currently processing the permit application. The system must notify the Department of any change in the chemicals or the manufacturers, and resultant modifications of its operations plan.

System Production

The table below summarizes the total amount of water produced by the Pajaro CSD system for 2008.

	Water Produced (million gallons)
Maximum Day	0.54
Maximum Month (July)	11
Annual Total	104.5

System Sources

The Pajaro CSD water system is supplied by two groundwater wells, hereafter referred to as Well 01 (PS Code: 2710020-001) and Well 02 (PS Code: 2710020-002). Well 01 is on standby status and Well 02 is the normal source of water supply.

WELL 02 (Active Source)

Well 02 was drilled in 1986 to a depth of 1200 feet and has a 14 inch diameter steel casing to 600 ft depth. The distance to the highest perforations is 450 ft. The total length of the screened interval is 145 ft. The annular seal extends from ground surface to 420 ft depth. Gravel packing is provided from 450 to 595 ft. A concrete surface seal that is 6 ft x 10 ft extends 20 inches above the ground. Well 02 is equipped with a water lubricated turbine pump with a capacity of 1600 gpm that discharges directly to the storage tank at about 45 feet horizontal distance. The well discharge line is equipped with an air release valve and a check valve. The raw water sample tap is located downstream of the check valve but is about 15 feet upstream of the chlorine injection port. Raw water samples are collected only when the well pump is running. Well 02 pumps directly into the Railroad Avenue storage tank and is controlled by a float switch in the storage tank. The well is housed in a metal building that is 6 ft x 6 ft in size. There is adequate drainage away from the well. The well is enclosed by a security fence.

WELL 01 (Standby Source)

Well 01 is classified as a standby source due to high levels of iron and manganese. Well 01was drilled in 1986 to a depth of 600 feet and has a 14-inch steel casing to 600 ft depth. The distance to the highest perforations is 430 ft. The total length of the screened interval is 150 ft. The annular seal extends to 415 ft. Gravel packing is provided from 430 to 595 feet. A concrete seal that is 13.5 ft x 10 ft extends 27 inches above the ground. Well 01 is equipped with a water lubricated turbine pump with a capacity of 800 gpm that discharges to the distribution system. The well discharge line is equipped with an air release valve and a check valve. The air release valve vent must be screened to prevent insects and rodents from entering the valve. The raw water sample tap is located downstream of the check valve. The raw water is not disinfected prior to distribution. There is adequate drainage away from the well. Well 01 is operated manually and used only in emergencies. However, the well pump is turned on periodically to flush the well and above ground piping system and the water wasted to the nearby storm drain. There is adequate drainage away from the well. The well site is located behind the District's office, next to an agricultural field and is enclosed by a security fence.

The system does not purchase water. The Department conducted the Drinking Water Source Assessment and Protection (DWSAP) program source assessments for both wells in July 2001. A copy of the assessment reports were provided to P/SM CSD at that time.

Treatment

The raw water from Well 02 is disinfected prior to storage and distribution. The raw water from Well 01 is not disinfected prior to distribution. The disinfection treatment system for Well 02 uses *Trichloroisocyanuric Acid* tablets to supply chlorine when dissolved in water. The system must ensure that the trade designation of the chemical as provided by the manufacturer is NSF/ANSI standard 60 certified and notify the Department of any changes in the product, manufacturer and treatment plant operation. The chlorine tablet feeder is an Arden Industries Model 22-100TRI Venturi Injector. While the manufacturer did not indicate the specific chlorine feed capacity, it did indicate that the capacity is well above the requirements for this water system. The water used to dissolve the tablets and create the chlorine solution is drawn from the well discharge line. As the water flows through the tablet feeder, it dissolves the tablets and thus prepared solution is stored in a clear calibrated container under the feeder. Concentration of solution can be controlled by adjusting the contact time within the tablet feeder. The level of tablets inside the feeder is maintained as close as possible to the marked outlet inside the feeder. During normal operation it takes about one scoop a week to maintain that level. The prepared solution is then injected into the drawn water as it is pumped back to the well discharge line about two feet downstream. The treatment feed system is housed in a 4 ft x 6 ft fiberglass building.

Storage

The system has one above ground steel storage tank that receives water from Well 02 and has a nominal capacity of 610,000 gallons. The actual storage capacity is approximately 600,000 gallons. It also has three hydropneumatic pressure tanks that add a small amount of additional storage but serve mainly to maintain pressure in the distribution system. The following table shows the tank details:

Name	Material	Year Installed	Last Inspection	Last Cleaned	Capacity (gallons)
Railroad Ave	Steel	1986	2005	2003	610,000
Pressure Tank (Near Well 2)	Steel	1986	2005		14,000
Pressure Tank (Near Well 2)	Steel	1986	2005		8,000
Pressure Tank (Near Well 1)	Steel	1986	2005		6,000

The Railroad Avenue storage tank hatch and the ladder access to the top of tank are locked. The roof vent is screened. The screen on the roof vent is broken and must be replaced to prevent possible contamination. The hatch interior, the top of the interior ladder and the roof vent are badly corroded and need to be cleaned and recoated per AWWA standards. It is strongly recommended by the Department that the tank interior be thoroughly inspected for any structural damage caused by the corrosion. If not repaired over time, this problem may result in a sanitary hazard. It is recommended to install screens on the air release valves for the pressure tanks and also on the tanks' drain line outlets. One sample tap is located at upstream of the tank and another one near the tank bottom near downstream pipe. All the tanks, pumps and the treatment facility are enclosed by a security fence. Alarms are installed on the tank access ladder and hatch, to detect any intrusion and remotely warn the staff.

Distribution System

The Pajaro distribution system consists of 4", 6", 8", and 10" C-900 PVC mains. The district has up to date distribution system maps. Service lines are mostly copper pipes or blue polybutylene pipes. There are no known lead service lines in the distribution system. The distribution system has one pressure zone. The system pressure is maintained between 60 and 80 psi.

Cross Connection Control Program

The system has a formal Cross Connection Control Program. The system conducts a cross-control survey annually. The system currently has 75 Backflow Prevention Devices (BPD) installed and one air gap. All the system's operators are certified backflow prevention device testers. All BPDs are tested on an annual basis. The district requires customers to test the devices once per year and to repair or replace the failed devices within a specific deadline depending upon the severity of the hazard, with options to either hire the district to do the work or to hire a certified tester/repairer/installer of backflow prevention devices.

Valve Maintenance Program

The system has 96 valves that range from 6 to 10 inches in size. The system has a valve maintenance program in place which includes exercising valves at least once per year. Valves are also checked and exercised during repairs and service calls.

Dead-End Flushing Program

The system has 9 dead-ends, three with blow-offs. The water main flushing program consists of flushing all water mains twice per year (6 months apart). The Pajaro system has multiple dead end lines that are flushed twice per year during the routine flushing program, but the system may do it more frequently if there are any water quality complaints.

Annual Reporting

The system submitted the 2008 Annual Report to the Drinking Water Program and the 2009 Consumer Confidence Report (CCR) and CCR Certification. The most recent Emergency Notification plan on file is dated March 9, 2007. However, it is recommended to revise the Emergency Notification Plan to include door to door notification. On May 20, 2010, the Department requested for an updated Emergency Notification Plan and provided a partially completed form with updated Department contacts and telephone numbers. The system **must** complete and submit the attached Emergency Notification Plan. On April 10, 2010 the Department provided written instructions to complete the Electronic Annual Report for calendar year 2009. To date, the Department has not received 2009 Annual Report for the Pajaro water system. The system must go to the DRINC Portal at http://drinc.ca.gov and submit the 2009 Electronic Annual Report.

Water Quality Monitoring

Bacteriological Monitoring

Source Monitoring

Well 02: Pajaro CSD water system is required to monitor Well 02 for bacteriological quality on a quarterly basis since the raw water is disinfected prior to storage and distribution. A review of the raw water bacteriological analysis results submitted to the Department for the period of March 2008 through March 2010 shows the system has complied with this requirement. All analysis results submitted for this sampling period were absent of total coliform and E. coli bacteria.

Well 01(standby): Pajaro CSD water system is required to monitor Well 01 for total coliform and E. coli prior to placing the well into service. A bacteriological sample must be collected after flushing the well and before placing the well into service and analyzed for total coliform and E. coli by an enumeration method. The sample results must be submitted to the Department as part of the notification for use of the standby source. The most recent analysis result submitted to the Department was for a sample collected on December 8, 2009. The sample results were absent for Total Coliform and E. coli.

Distribution System Monitoring

The most recent Bacteriological Sample Siting Plan (BSSP) on file was received by the Department on July 10, 2006. The system is required to collect seven (7) routine samples in the distribution system per month and test them for coliform bacteria. Distribution system monitoring results indicate an absence of coliform for at least two years. The system is in compliance with the distribution system bacteriological quality monitoring. In order to comply with the federal Groundwater Rule, system must submit the attached updated BSSP to the Department.

Chemical

Source Monitoring

Inorganic: The system is required to monitor Well 02 for Title 22 regulated inorganic chemicals once every three years with the exception of cyanide, which is waived. Inorganics were last monitored for Well 02 in 2009. All inorganic constituents were below one half their MCLs or not detected. The next inorganic monitoring for Well 02 is due in **2012**. For Well 01, since it is a standby well, inorganic monitoring is required once every nine years. The last round of inorganic monitoring was in 2004. All Well 01 inorganic constituents were below their respective DLRs in this round of sampling. The next inorganic monitoring is due in **2013**.

Asbestos: For community water systems, source monitoring for asbestos is required once every nine years for hard rock wells. Asbestos was not detected in a sample collected from Well 02 on April 14, 2003. Since Well 02 is not a hard rock well or vulnerable to asbestos contamination by other means, the system is eligible to apply for an asbestos monitoring waiver for this source. The system must complete and submit to the Department the waiver request form (Enclosed). Once received, the Department will grant a waiver for asbestos monitoring at the source. For Well 1 (*standby*) waiver for asbestos monitoring at the source pursuant to Section 64432.2(c) has been granted by the Department. Additional monitoring for asbestos at Well 01 is not required.

Nitrate/Nitrite: The system is required to monitor nitrate on an annual basis for Well 02 and once every nine years for Well 01. Nitrate concentrations in these sources are below the DLR of 2.0 mg/L. The next nitrate monitoring for Well 02 is due in 2011 and for Well 01, it is due in **2019**. The system is required to monitor nitrite once every three years in Well 02 and once every nine years in Well 01. Nitrite was last monitored in Well 02 on September 22, 2009 and on April 28, 2004 for Well 01. Nitrite was not detected in both wells at that time. The next nitrite monitoring for Well 02 is **2012** and **2013** for Well 01.

In addition, Well 1 must be monitored for nitrate prior to bringing the well into service.

General Minerals/General Physical Standards (GPs): The system is required to monitor Title 22 regulated GPs once every three years for Well 02 and once every nine years for Well 01. GPs were last monitored in Well 02 on September 22, 2009 and on December 8, 2009 in Well 01. All constituents with secondary MCLs were below their respective MCLs except for manganese in Well 01. In Well 01, manganese averaged 0.355 mg/L in the last two samples (2008 – 2009). Since Well 01 is a standby well, no action is required for this constituent. The next GP monitoring for Well 02 is due in **2012** and in **2013** for Well 01.

Radiological: The system completed initial monitoring requirement for radionuclides at Well 02 in 2006 including four quarterly samples for radium-228. Gross alpha monitoring frequency for this source is reduced to once every nine years. Monitoring for uranium, radium-226, and radium-228 is not required unless the next gross alpha result exceeds 5 pCi/L. The next monitoring for gross alpha analysis for Well 02 is due in **2015**. For Well 01, radiological monitoring is required once every nine years. Well 01 was monitored for gross alpha twice in 2004 and for radium-228 once in 2006. Sample results are below their respective DLRs. The next radiological monitoring required for Well 01 is due in **2013**.

Volatile Organic Compounds (VOCs): The system is required to monitor Well 02 for Title 22 regulated VOCs once every three (3) years. For Well 02, the most recent monitoring for all VOCs was completed on June 16, 2009. No VOCs were detected at that time. The next monitoring is due in **2012**. For Well 01, VOC monitoring is required once every nine (9) years. VOCs were last monitored in Well 01 in 2004. No VOCs were detected in that sample. The next VOC monitoring for Well 01 is due in **2013**.

Methyl-Tert-Butyl-Ether (MTBE): Initial monitoring for MTBE consists of four consecutive quarterly samples followed by two years of annual samples. The system completed the four quarterly samples for Well 02 in 2003 and 2004. MTBE was not detected. Annual samples were collected in 2005 and 2006, also resulting in no detection of MTBE. The most recent monitoring was completed in June 2009. The MTBE monitoring frequency for Well 02 has been reduced to once every three (3) years. The next monitoring is due in **2012**. For Well 01 as a standby source, two samples were collected in 2004, and no MTBE was detected. The repeat monitoring frequency for Well 01 is once every nine (9) years. The next monitoring is due in **2013**.

The Department completed an MTBE vulnerability assessment for Well 01 and Well 02 based on activities surrounding the well. Both wells are located within 1,500 feet of activities that are considered potential sources of MTBE contamination, including fleet/trucks terminal, gas station, auto body shop, and railroad yard. Both Well 01 and Well 02 have been determined to be vulnerable to MTBE. MTBE has not been detected in the water from either well.

Synthetic Organic Compounds (SOCs): The system is required to monitor Well 02 for the following SOCs: alachlor, atrazine, bentazon, carbofuran, diquat, simazine, and 2,4-dichlorophenoxyacetic acid (2,4-D). The required SOC monitoring frequency is two (2) consecutive quarters every three (3) years.

The water system submitted a waiver request form for SOC monitoring dated July 24, 2008. The Department granted a "use" and/or "susceptibility" monitoring waiver for most Title 22 regulated SOCs except for alachlor, atrazine, bentazon, carbofuran, diquat, simazine, and 2,4-D. The term of the waiver ends on **December 31, 2010**.

The water system monitored Well 02 for SOCs in September 2007. Only one quarterly sample was collected in 2007. No SOCs were detected at that time. Although none were detected at that time, the system is <u>not</u>

compliant with the monitoring frequency required for these regulated SOCs. The next monitoring must be completed for **4th quarter 2010** and **1st quarter 2011,** and include all required SOCs listed above.

For Well 01, one (1) SOC sample is required every nine (9) years. The last SOC monitoring for Well 01 was conducted in 2004. No SOCs were detected in that sample. Next monitoring is due in **2013.**

Distribution System Monitoring

Lead and Copper: The system is required to monitor for lead and copper at 20 sample sites per reduced monitoring requirements. The samples must be taken during the month of June, July, August, or September. The system's lead and copper monitoring results and schedule is shown in the table below. The next lead and copper monitoring must be conducted in the month of **June, July, August or September of 2010.**

Sample Set	Reduced Monitoring Approved?	Sample Date	No. of Samples	90th Percentile Lead (mg/L) AL=0.015 mg/L	90th Percentile Copper (mg/L) AL=1.3 mg/L
1st 6 Month	N	9/27/1995	30	<0.005	0.24
2nd 6 Month	N	3/21/1996	30	<0.005	0.21
1st Annual	Not Required	Not Required			
2nd Annual	Not Required	Not Required			
1st Triennial	Yes	September 1999	20	Failed to monitor	Failed to monitor
1st Triennial	Yes	9/25/2002	30	0.005	0.09
2nd Triennial	Yes	September 2005	20	Failed to monitor	Failed to monitor
2nd Triennial	Yes	September 2007	20	<0.005	0.143

Disinfectants/Disinfection Byproducts: Pajaro CSD is required to monitor the distribution system for disinfection byproducts since the water from Well 02 is chlorinated prior to storage and distribution. The most recent Disinfectants/Disinfection Byproducts Monitoring Plan on file is dated July 10, 2005. Routine Total Trihalomethane (TTHM) and Haloacetic Acid (HAA5) monitoring consists of collecting one sample per year during the month of September. Designated sample location is 26 Jonathan Street. In 2009, the TTHM and HAA5 results were 2.3 µg/L and ND, respectively. The next TTHM and HAA5 monitoring must be conducted in **September 2010** and the results reported to the Department by **October 10, 2010.** Routine chlorine residual monitoring is performed at the same frequency and sampling locations as the distribution system bacteriological monitoring. The running annual average for chlorine residual is 0.3 mg/L as of the 2nd quarter of 2010. The system is compliant with the Maximum Disinfectant Residual Level of 4.0 mg/L.

Asbestos: The distribution system does not contain asbestos-cement mains. Asbestos monitoring in the distribution system is waived.

Overall System Appraisal

The water produced by the system's primary well is hard but otherwise of good quality. The system has adequate capacity and can meet current demand. The in-house certified operators appear to be conscientious and, except for the corrosion in the main storage tank, are operating and maintaining the system in good condition. The main storage tank internal corrosion is a matter of serious concern to the Department, which in case of failure, may render the distribution system without sufficient amount of water for a significant period of time. Required monitoring is current except for Well 02, SOCs monitoring must be completed for two consecutive quarters in 2010.

Deficiencies

1. The Pajaro CSD water system must submit updated operations plan for the Pajaro disinfection facility to the Department by **November 1, 2010.**
2. The system must submit updated BSSP (attached) by **November 1, 2010** to include monitoring required by Ground Water Rule.
3. The system must inspect the 610,000 gal. main storage tank on a regular basis and periodically evaluate its deterioration due to internal corrosion. The inspection must include the evaluation of the tank overflow pipe for possible deterioration, since the pipe is underground and its outlet is submerged in the groundwater. Submit proof of monthly inspection by **November 1, 2010.** It is recommended by the Department that the tank overflow pipe outlet be raised well above the water level and faced downward.
4. Since the underground tank(s) under the booster pumps are inaccessible for inspection, there is a possibility of potential contaminant from surrounding high ground water to enter through any possible cracks in the tank walls. Therefore, in the absence of a reliable inspection method, the system must sample water from the downstream of the booster pumps once per month and test for total coliform. The samples must be labeled as "other" and the results submitted by the tenth day of the following month. The samples can not be used for compliance with the Total Coliform Rule. First results must be submitted by **November 10, 2010.**
5. A screen must be installed on outlet of the overflow pipe of the 610,000 gal. storage tank, to prevent rodent or insect intrusion. Submit proof by **November 1, 2010.**
6. The system must eliminate the corrosion on the 610,000 gal. storage tank interior and replace the screen on the roof vent. If the tank fails the water system is at risk of outages. Submit proof of screen installation by **November 1, 2010** and a plan and schedule for correcting the corrosion by **November 30, 2010.**
7. The system must monitor for SOCs for the **4th quarter 2010** and **1st quarter 2011.**
8. It was observed during the inspection that there is a railroad yard located within a 1500 feet radius of Well 02. The system must provide to the Department a method of notification set forth by the rail road authority and the water system, in case any possible hazardous spill occurs, by **November 1, 2010.**
9. Screen must be installed on the air release valve near Well 01. Submit proof by **November 1, 2010.**
10. The system must submit the updated Emergency Notification Plan and Electronic Annual Report for the calendar year 2009, by **November 1, 2010.**
11. The system must ensure that the Drinking Water Treatment Chemicals are NSF/ANSI standard 60 certified. The system must notify the Department of any changes in the chemicals or manufacturers, and treatment plant operation. The system must submit the proof that the trade designation of the product

Trichloroisocyanuric Acid, as provided by the manufacturer, is listed on the NSF/ANSI 60 product and service list, by **November 1, 2010.**

Recommendations

1. It is recommended by the Department that water district should develop a Capital Improvements Plan/Equipment Replacement Plan. The Pajaro Community Service District should develop a budget that includes reserves for emergencies (e.g., emergency reserve funds for unplanned equipment repair or replacement), and reserves for Capital Improvement Projects.

Report Prepared By:

Shaminder Kler
Sanitary Engineer

Attachments:
1. Inspection Photos
2. MTBE Vulnerability Assessment form
3. New GWR BSSP
4. Asbestos Monitoring Waiver Request form
5. Emergency Notification Plan form

APPENDIX B

Tank Inspection Report

Inland Potable Services, Inc.

16297 E. Crestline Lane
Centennial, CO 80015
Phone: 303-400-4220
Fax: 303-400-4215

Inspection Report for

Pajaro/Sunny Mesa CSD
Royal Oaks, CA

600KG Steel On-Grade Tank

Date Completed: February 2, 2013

Commercial Dive Team:

Diver –James Bingham
Dive Controller –Jason Gardner
Tender –Jeff Roberts

Scope of Work:

Our team completed sediment removal using underwater vacuum equipment. Sediment depth averaging 1/8 inch (iron & manganese) was removed from tank floor. When the cleaning process was finished, a full visual inspection was performed of the tank interior and all interior fixtures. The team also performed a full visual inspection of the tank exterior and all attached fixtures. The details of the inspection findings are included in the report below.

Summary of the Inspection:

Exterior Inspection

1. There was good access to the tank. (In a gated area)
2. The ladder was found secure, OSHA approved and in good condition with biological growth, minor de-lamination, heavy oxidation and less than 1% surface corrosion noted.
3. The roof was found in good condition with heavy biological growth & oxidation, low spots and 1% surface corrosion noted.
4. The hatch was found locked with no gasket present and in poor condition with biological growth, de-alloying, heavy oxidation, de-lamination and 33% surface corrosion noted.
5. The walls were found in good condition with biological growth and 1% surface corrosion noted.
6. The vent was found in fair condition with heavy oxidation, minor de-lamination and 33% surface corrosion noted and a screen in place.
7. The manways were found secure and in good condition with biological growth, de-alloying, de-lamination and 1% surface corrosion noted.

Interior Inspection

1. The inlet and outlet were found in good condition with heavy staining and less than 1% surface corrosion noted.
2. The ladder, overflow, support column and floor were found in fair condition with heavy staining, rust nodules, blistering, pitting and surface corrosion noted.
3. The manways were found in good condition with heavy staining & blistering and less than 1% surface corrosion noted.
4. The interior walls were found in fair condition with heavy staining, micro blisters, pitting and 1% surface corrosion noted.
5. The interior roof was found in good condition with concentrated cell corrosion, corrosive staining, de-alloying and 2% surface corrosion noted. The support beams were also found to be warped.

Recommendations:

1. Install a gasket on the access hatch.
2. Schedule time for a blast and recoat.
3. Schedule time to clean and inspect every 3-5 years per AWWA recommendations.

Key

Excellent – Like new, no repairs needed
Good – Cosmetic problems, repair if utility wants
Fair – Minor problems, repairs needed
Poor – Major problems, fix now

Inland Potable Services, Inc.

Exterior Inspection Report

Access Ladder Condition

Ladder Type: Steel
Coating Condition: Fair
Corrosion Present? Y ☒ N ☐
Seams/Welds Condition: Excellent
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Stand Off Supports Condition: Good
Safety Climb Type: Cage
Safety Climb Condition: Excellent
Is Top Of Tank Easily Accessible? Y ☒ N ☐
Is The Ladder and Safety Climb **OSHA** Approved? Y ☒ N ☐

Summary: The ladder was found secure, OSHA approved and in good condition with biological growth, minor de-lamination, heavy oxidation and less than 1% surface corrosion noted.

Roof Condition

Coating Condition: Good
Corrosion Present? Y ☒ N ☐
    Percentage: 1%
Seams/Welds Condition: Excellent
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Low Spots Present? Y ☒ N ☐
Holes in Roof? Y ☐ N ☒
Cathodic Protection Plates Present? Y ☒ N ☐
    Sealed Edges: Y ☒ N ☐ N/A ☐
    Loose Plates? Y ☐ N ☒ N/A ☐
    Missing Plates? Y ☐ N ☒ N/A ☐

Summary: The roof was found in good condition with heavy biological growth & oxidation, low spots and 1% surface corrosion noted.

Access Hatch Condition

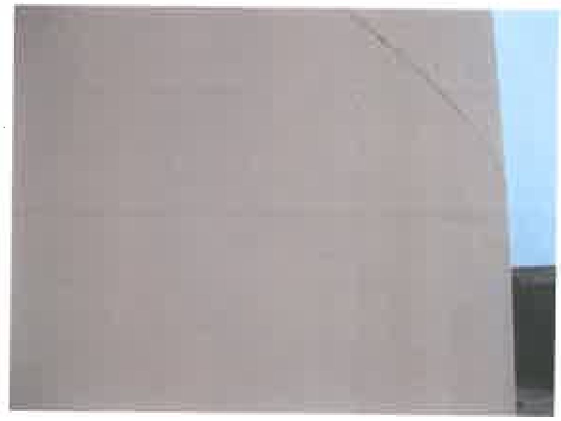
Coating Condition: Poor
Corrosion Present: Y ☒ N ☐
Seams/Welds Condition: Good
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Hatch Size: 2 foot square
    Hatch Locked? Y ☒ N ☐
    Hinge Condition: Good
Gasket Present? Y ☐ N ☒
    Intact? Y ☐ N ☐ N/A ☒
Insects, Dirt Or Debris Present Under Hatch? Y ☐ N ☒

Summary: The hatch was found locked with no gasket present and in poor condition with biological growth, de-alloying, heavy oxidation, de-lamination and 33% surface corrosion noted. Recommend a gasket.

Wall Panel Condition

Coating Condition: Good
Corrosion Present? Y ☒ N ☐
Percentage: 1%
Seams/Welds Condition: Excellent
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐
Dents Present? Y ☐ N ☒
Holes Present? Y ☐ N ☒

Summary: The walls were found in good condition with biological growth and 1% surface corrosion noted.



Vent Condition

Coating Condition: Fair
Corrosion Present: Y ☒ N ☐
Percentage: 33%
Seams/Welds Condition:
Oxidation Present? Y ☒ N ☐
De-lamination Present? Y ☒ N ☐

Screen in Place? Y ☒ N ☐
Condition: Good
All Openings Sealed? Y ☒ N ☐
Cap Condition: Good

Summary: The vent was found in fair condition with heavy oxidation, minor de-lamination and 33% surface corrosion noted and a screen in place.

Foundation Condition

Foundation Exposed? Y ☒ N ☐
Anchor Bolts Present? Y ☐ N ☒
Corrosion on Anchor Bolts Present? Y ☐ N ☐ N/A ☒
Anchor Bolts Loose? Y ☐ N ☐ N/A ☒

Cracking Noted In Foundation? Y ☐ N ☒
Spalling Noted? Y ☐ N ☒

Summary: The foundation was found in excellent condition.

Manway Condition	
Coating Condition: Both Fair Weld/Seam Condition: Both Excellent Corrosion Present? Y ☒ N ☐ Percentage: 1% Pitting Noted In Metal? Y ☐ N ☒ Depth: N/A Summary: The manways were found secure and in good condition with biological growth, de-alloying, de-lamination and 1% surface corrosion noted.	
	

Inland Potable Services, Inc.
Interior Inspection Report

Inlet and Outlet Condition

Common Inlet/Outlet? Y ☐ N ☒ Location: N/A
If No:
Inlet Location: 4:30 o'clock
Coating Condition: Good
Weld/Seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: less than 1%
Pitting Noted In Metal? Y ☐ N ☒
Depth: N/A

Summary: The inlet was found in good condition with heavy staining and less than 1% surface corrosion noted.

Common Inlet/Outlet? Y ☐ N ☐ Location:
If No:
Outlet Location: 10:30 o'clock
Coating Condition: Good
Weld/Seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: less than 1%
Pitting Noted In Metal? Y ☐ N ☒
Depth: N/A

Summary: The outlet was found in good condition with heavy staining and less than 1% surface corrosion noted.

Ladder Condition

Ladder Location: 12 o'clock
Coating Condition: Fair
Weld/Seam Condition: Fair
Supports Condition: Fair
Corrosion Present? Y ☒ N ☐
Percentage: 3%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The ladder was found in fair condition with heavy staining, rust nodules, heavy blistering, pitting and 3% surface corrosion noted. The cage was found in poor condition.

Manway Condition

Manway Locations: 3 o'clock & 8 o'clock
Coating Condition: Both Fair
Weld/Seam Condition: Both Excellent
Corrosion Present? Y ☒ N ☐
Percentage: less than 1%
Pitting Noted In Metal? Y ☐ N ☒
Depth: N/A

Summary: The manways were found in good condition with heavy staining & blistering and less than 1% surface corrosion noted.

Overflow Condition

Overflow Location: 9:30 o'clock
Coating Condition: Fair
Weld/Seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: 1%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The overflow was found in fair condition with heavy staining & blistering, corrosive staining, rust nodules, pitting and 1% surface corrosion noted.

Wall Panel Condition

Coating Condition: Fair
Welds/seam Condition: Excellent
Corrosion Present On Panel? Y ☒ N ☐
Percentage: 1%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The interior walls were found in fair condition with heavy staining, micro blisters, pitting and 1% surface corrosion noted.

Roof Condition

Coating Condition: Fair
Welds/seam Condition: Good
Corrosion Present On Panels? Y ☒ N ☐
Percentage: 2%
Metal De-alloying Noted? Y ☒ N ☐
Percentage: less than 1%

Summary: The interior roof was found in good condition with concentrated cell corrosion, corrosive staining, de-alloying and 2% surface corrosion noted. The support beams were found to be warped.

Support Column Condition

Coating Condition: Fair
Welds/seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percent: 2%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

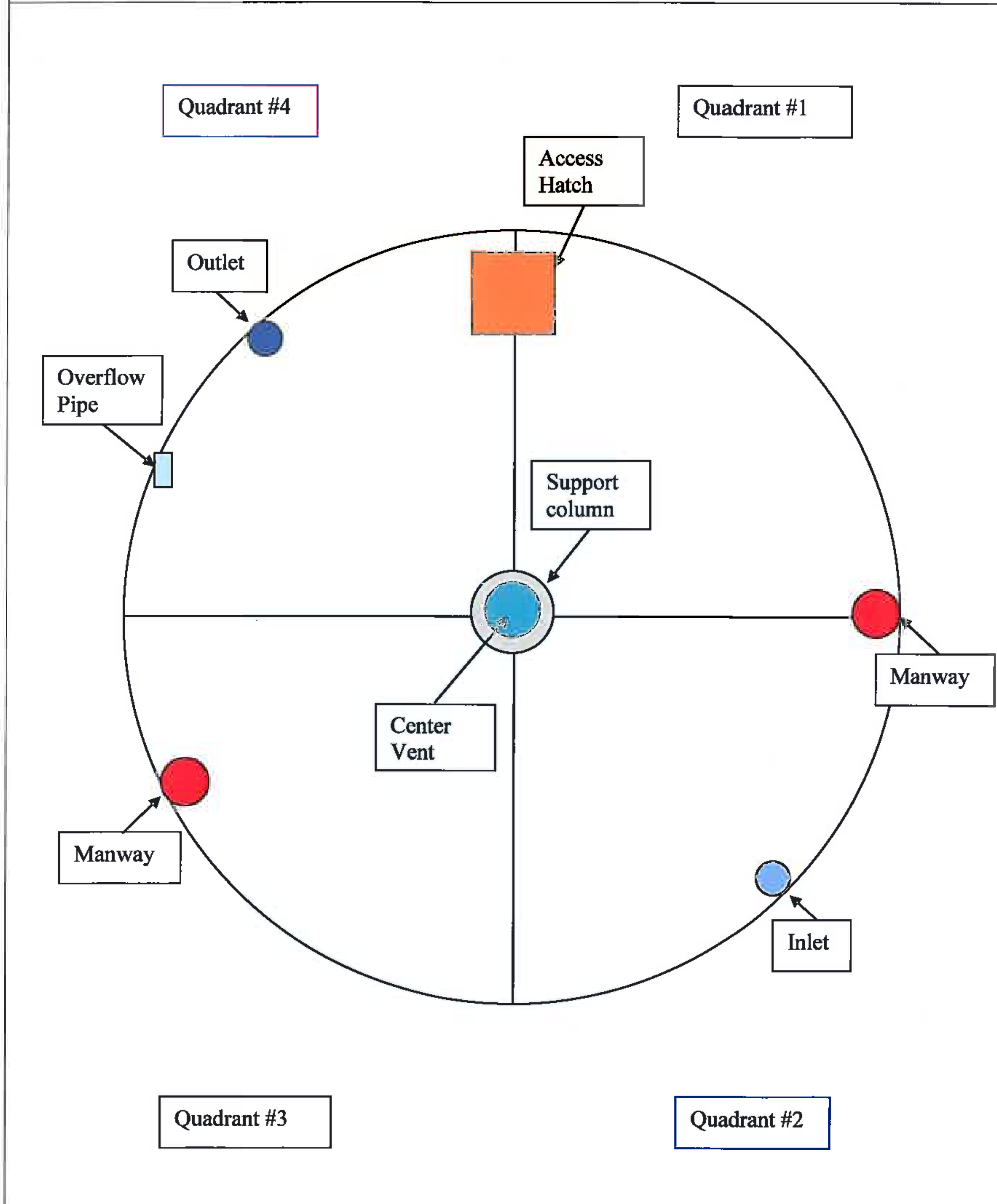
Summary: The support column was found secure and in fair condition with heavy staining, rust nodules, pitting and 2% surface corrosion noted.

Floor Condition

Coating Condition: Fair
Welds/seam Condition: Excellent
Corrosion Present? Y ☒ N ☐
Percentage: 2%
Pitting Noted In Metal? Y ☒ N ☐
Depth: 1/8 inch

Summary: The floor was found in fair condition with heavy staining, rust nodules, blistering, pitting and 2% surface corrosion noted.

Tank Layout


Quadrant #4
Quadrant #1
Access Hatch
Outlet
Overflow Pipe
Support column
Manway
Center Vent
Manway
Inlet
Quadrant #3
Quadrant #2

APPENDIX C

Cost Estimates

OPINION OF PROBABLE CONSTRUCTION COST

KENNEDY/JENKS CONSULTANTS

Project: Pajaro Sunny Mesa - Prop 84 Funding

Building, Area: Rehabilitate Existing 600,000 Gallon Tank

Prepared By: NEP
Date Prepared: 25-Jan-13
K/J Proj. No. 0985019

Estimate Type:
- [x] Conceptual
- [ ] Preliminary (w/o plans)
- [ ] Design Development @

- [ ] Construction
- [ ] Change Order

_____ % Complete

Current at ENR ______
Escalated to ENR ______
Months to Midpoint of Construct 18

Spec. No.	Item No.	Description	Qty	Units	Materials		Installation		Sub-contractor		Total
					$/Unit	Total	$/Unit	Total	$/Unit	Total	
		Sandblast and Recoat Exterior of Existing Tank	15,750	SF	4.00	63,000	5.00	78,750			141,750
		Structural Repair of Interior of Tank	1	LS	10,000.00	10,000	20,000.00	20,000			30,000
		Sandblast and Recoat Interior of Existing Tank	15,750	SF	5.00	78,750	9.00	141,750			220,500
		Repair/Replace Existing Tank Appurtenances	1	LS	9,000.00	9,000	20,000.00	20,000			29,000
		Subtotals				160750.00		260500.00			421250.00
		Division 1 Costs	@								
		Subtotals				160750.00		260500.00			421250.00
		Taxes - Materials Costs	@	8.25%		13261.88					13261.88
		Subtotals				174011.88		260500.00			434511.88
		Taxes - Labor Costs	@								
		Subtotals				174011.88		260500.00			434511.88
		Contractor Markup for Sub	@	12%							
		Subtotals				174011.88		260500.00			434511.88
		Contractor OH&P	@	15%		26101.78		39075.00			65176.78
		Subtotals				200113.66		299575.00			499688.66
		Estimate Contingency	@	15%							74953.30
		Subtotals									574641.95
		Escalate to Midpoint of Construct	@	3%							25858.89
		Estimated Bid Cost									600,500.84
		Total Estimate									600,510
		Total Estimate									**600,000**

OPINION OF PROBABLE CONSTRUCTION COST

KENNEDY/JENKS CONSULTANTS

Project: Pajaro Sunny Mesa - Prop 84 Funding

Building, Area: Option A - 600,000 Gallon Welded Steel Tank

Prepared By: NEP
Date Prepared: 25-Jan-13
K/J Proj. No. 0985019

Estimate Type:
- [x] Conceptual
- [ ] Preliminary (w/o plans)
- [ ] Design Development @
- [ ] Construction
- [ ] Change Order

% Complete:

Current at ENR:
Escalated to ENR:
Months to Midpoint of Construct: 18

Spec. No.	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total
		Remove and Stockpile Existing Stored Materials	1,000	CY	0.50	500	5.00	5,000			5,500
		Wick Drains	1	LS	15,000.00	15,000	20,000.00	20,000			35,000
		Tank Base, Exist. Materials placed and compacted	750	CY	0.50	375	7.50	5,625			6,000
		Sand Layer Below Tank to allow Drainage	240	CY	10.00	2,400	30.00	7,200			9,600
		600,000 Gallon Tank - 80' Diamter	1	EA	350,000.00	350,000	350,000.00	350,000			700,000
		Misc Piping, Valves and Appurtenances	1	LS	75,000.00	75,000	75,000.00	75,000			150,000
		Misc. Site Work	1	LS	25,000.00	25,000	25,000.00	25,000			50,000
		Subtotals				468275.00		487825.00			956100.00
		Division 1 Costs	@								
		Subtotals				468275.00		487825.00			956100.00
		Taxes - Materials Costs	@	8.25%		38632.69					38632.69
		Subtotals				506907.69		487825.00			994732.69
		Taxes - Labor Costs	@								
		Subtotals				506907.69		487825.00			994732.69
		Contractor Markup for Sub	@	12%							
		Subtotals				506907.69		487825.00			994732.69
		Contractor OH&P	@	15%		76036.15		73173.75			149209.90
		Subtotals				582943.84		560998.75			1143942.59
		Estimate Contingency	@	15%							171591.39
		Subtotals									1315533.98
		Escalate to Midpoint of Construct	@	3%							59199.03
		Estimated Bid Cost									1,374,733.01
		Total Estimate									1,374,740
		Total Estimate									**1,400,000**

OPINION OF PROBABLE CONSTRUCTION COST

KENNEDY/JENKS CONSULTANTS

Project: Pajaro Sunny Mesa - Prop 84 Funding

Building, Area: Option B - 1,200,000 Gallon Concrete Tank

Prepared By: NEP
Date Prepared: 25-Jan-13
K/J Proj. No. 0985019

Current at ENR
Escalated to ENR
Months to Midpoint of Construct 18

Estimate Type:
- [x] Conceptual
- [ ] Preliminary (w/o plans)
- [ ] Design Development @
- [ ] Construction
- [ ] Change Order

% Complete

Spec. No.	Item No.	Description	Qty	Units	Materials $/Unit	Materials Total	Installation $/Unit	Installation Total	Sub-contractor $/Unit	Sub-contractor Total	Total
		Remove and Stockpile Existing Stored Materials	1,000	CY	0.50	500	5.00	5,000			5,500
		Wick Drains, 4' Spacing	1	LS	30,000.00	30,000	25,000.00	25,000			55,000
		Tank Base, Exist. Materials placed and compacted	1,200	CY	0.50	600	7.50	9,000			9,600
		Sand Layer Below Tank to allow Drainage	400	CY	10.00	4,000	30.00	12,000			16,000
		1,200,000 Gallon Concrete Tank	1	EA	500,000.00	500,000	500,000.00	500,000			1,000,000
		Misc Site Piping Valves and Appurtenances	1	LS	75,000.00	75,000	75,000.00	75,000			150,000
		Misc Site Work	1	LS	25,000.00	25,000	25,000.00	25,000			50,000
		Demolish and Remove Existing 600,000 Gal Tank	1	LS	5,000.00	5,000	20,000.00	20,000			25,000
		Subtotals				640100.00		671000.00			1311100.00
		Division 1 Costs	@								
		Subtotals				640100.00		671000.00			1311100.00
		Taxes - Materials Costs	@	8.25%		52808.25					52808.25
		Subtotals				692908.25		671000.00			1363908.25
		Taxes - Labor Costs	@								
		Subtotals				692908.25		671000.00			1363908.25
		Contractor Markup for Sub	@	12%							
		Subtotals				692908.25		671000.00			1363908.25
		Contractor OH&P	@	15%		103936.24		100650.00			204586.24
		Subtotals				796844.49		771650.00			1568494.49
		Estimate Contingency	@	15%							235274.17
		Subtotals									1803768.66
		Escalate to Midpoint of Construct	@	3%							81169.59
		Estimated Bid Cost									1,884,938.25
		Total Estimate									1,884,940
		Total Estimate									1,900,000

	REHABILITATION	OPTION A	OPTION B
		600,000	1,200,000
Envir	$ -	$ 50,000	$ 50,000
Design	$ 60,000	$ 140,000	$ 180,000
Surveying	$ -	$ 15,000	$ 15,000
Geotech	$ -	$ 25,000	$ 25,000
CM	$ 80,000	$ 140,000	$ 170,000
Total	$ 140,000	$ 370,000	$ 440,000